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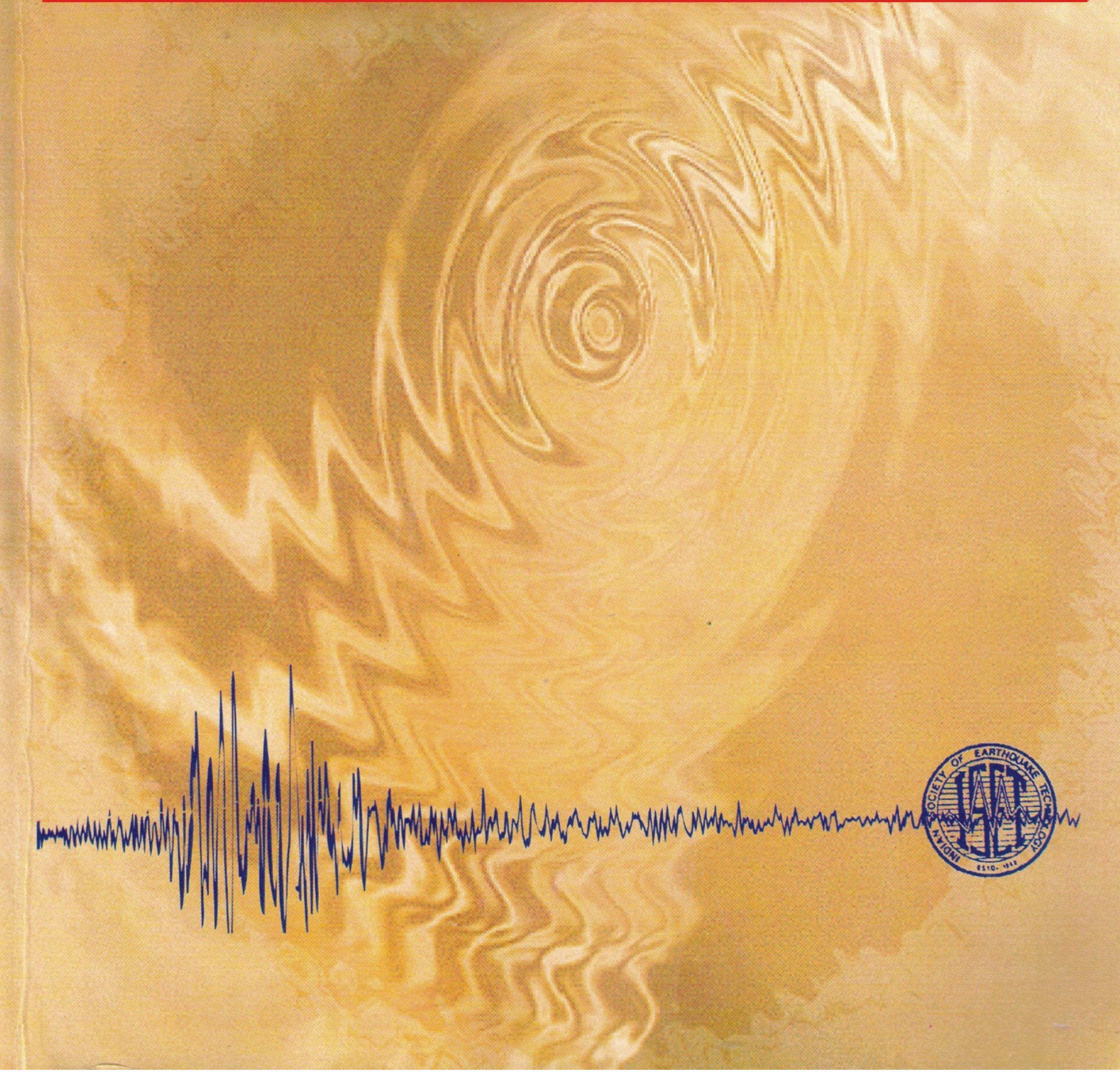

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ii. To disseminate knowledge in the field of earthquake technology dealing with seismological and engineering aspects.
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ISET JOURNAL OF EARTHQUAKE TECHNOLOGY

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CONTENTS

ISET Journal of Earthquake Technology, Paper No. 528, Vol. 51, No. 1-4, March-Dec. 2014, pp. 1–17

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SEISMIC DESIGN AND SAFETY ASPECTS OF DAMS

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ABSTRACT

The seismic safety of dams has assumed greater importance now because of associated widespread human and economic loss. Strong earthquakes affect large areas, subjecting many dams to strong ground shaking. This is particularly important for Himalayas, where large number of dams have been built or are being built or planned. The 2001 Bhuj earthquake affected about 245 dams, though mainly small dams, but the large scale dam failures calls for review of present design practices. More and more stringent criteria are now being prescribed for seismic design. The earlier methods adopted for seismic analysis of dams are either considered obsolete or even wrong today. There is thus an urgent need for systematic review of existing design techniques and reassessment of seismic safety of existing dams. ICOLD has recently revised its seismic design criteria (Bulletins 120, 123, 148) which are different from the earlier one (Bulletin 72). Also, USACE has also come out with guidelines on seismic design of dams (EM-1110-2-6050, 6051, 6053, etc.). This change shows that dams which were considered safe at the time of completion may not retain that certification even if maintained properly. The National Committee of Seismic Design Parameters (NCSDP) of India is also periodically reviewing its guidelines on seismic parameters to be adopted for dam design based on the state-of-the-art guidelines outlined in ICOLD, USACE and other publications. Dynamic analysis of dams, either using response spectra or time history, has now become almost mandatory for all the dams being designed or planned in India. The seismic coefficient/pseudo static method is considered deficient as it may not satisfy today's seismic criteria. However, a large pool of experience and performance data of dams exist which were designed by such methods and continue to perform their designed function. Their performance, as viewed from a more rigorous angle leads to additional cost and construction implications. Amalgamation of presently available dam sections with those needed for the modern approach is required so that while not compromising on safety, a panic reaction is also avoided. This paper discusses the present practices on seismic design and performance criteria for dams; conceptual and construction requirement for seismic requirement of concrete gravity and embankment dams and also the need for periodic review of seismic safety of existing dams.

KEYWORDS: Dams, Seismic design, Safety aspects, ICOLD

INTRODUCTION

The field of dam safety encompasses such diverse subjects as hydrological safety, hydraulic safety, hydro-mechanical safety and the structural safety of dam. The structural safety means design of a dam-following nationally and internationally accepted guidelines-for flood hazard, earthquake hazard, seepage, and various other hazards from the natural and man-made environment including site-specific and project-specific hazards. In recent years, the earthquake hazard has gained much importance in the dam industry. The seismic safety of dams has assumed greater importance now because of associated widespread human and economic loss. This is particularly important for Himalayas, where large number of dams have been built or are being built or planned.

Damage to dams and their appurtenant facilities may result from direct fault movement across the dam foundation or, more likely, from ground motion induced at the dam site by an earthquake located at some distance from the dam. However, as strong earthquakes may affect a large area, many dams may be subjected to strong ground shaking as in the case of the May 12, 2008 Wenchuan earthquake in China, where about 1803 dams and reservoirs –most of them were small earth dams while four dams had a height exceeding 100m-and 403 hydropower plants were damaged (Wieland and Chen, 2009). Also, during the 2001 Bhuj earthquake in Gujarat, India, 245 dams-mainly small embankment dams-had to be rehabilitated or strengthened after the earthquake. The latest earthquake which affected many dams was the March 11, 2011 Tohoku earthquake in Japan where one 18 m high embankment dam failed and 8 people lost their lives, while another 300 dams, subjected to earthquake shaking, had to be inspected.

The seismic design criteria and methods of dynamic analysis of dams have undergone substantial changes since the 1930s when earthquake actions have been introduced to their design. At that time, the earthquake hazard was confined to the effect of ground shaking, which was represented by a seismic coefficient. Typically, a value of 0.1 was used for most dams; and in exceptional cases slightly higher values were considered. However, the seismic coefficients thus applied had no clear physical relation with the design ground motions and the seismic hazard at the dam site. Moreover, the dynamic response was determined by a pseudo-static analysis, which does not account for the dynamic characteristics of the dam. As most existing dams built before the 1990s were designed against earthquakes using such seismic design criteria and/or methods of dynamic analysis-which are considered obsolete or even wrong today-the earthquake safety of these dams becomes somewhat uncertain if modern criteria are to be applied. This means that dams which were considered safe at the time of completion may not retain that certification even if maintained properly. It has to be assumed that a few of them may be structurally deficient; and consequently, there is a need for the systematic reassessment of earthquake safety of existing dams-at least those classified as large dams[1].

Today we have greater clarity on the seismic design criteria to be applied when a dam is subjected to ground shaking, and better methods of dynamic analyses have been developed which even allow for calculation of the inelastic seismic response of embankment and concrete dams. However, focussing solely on the dynamic analyses of dams may not suffice as it is not possible to make a dam with conceptual deficiencies to perform well during strong earthquakes by carrying out sophisticated dynamic analyses. Very often the conceptual and constructional guidelines are found to be more effective than analyses. Thus, the three-fold key challenges that we face today in the earthquake-resistant design and construction of large dams pertain to:

- Selection of ground motion parameters of the different design earthquakes based on site specific seismic hazard analyses, and selection of appropriate methods of seismic analysis;
- Observing conceptual guidelines and detailing recommendations for the earthquake-resistant design of dams; and
- Ensuring high quality of all construction works.

Further, it also needs to be underlined that a large pool of experience and performance data of dams exists-which were designed by methods seemingly obsolete today-and continue to perform their designed function. Their continued performance calls for greater justification for design revisions viewed from a more rigorous angle leading to additional cost and construction implications. Evidently, an amalgamation of presently available dam sections with those needed for the modern approach may be required so that while not compromising on safety, a panic reaction is also avoided.

SEISMIC HAZARDS AND SAFETY CONCEPT FOR DAMS

The earthquakes can cause multiple hazards including ground shaking, fault movements in dam foundations and reservoirs, rock falls, landslides, liquefaction, water waves in the reservoir etc. Effects such as water waves and reservoir oscillations are of lesser importance for the earthquake safety of a dam[2]. These surface waves can be compared with those caused by wind. Usually the main hazard, which is addressed in codes and regulations, is the earthquake ground shaking. It causes stresses, deformations, cracking, sliding, overturning, etc. However, even other hazards-which are normally not covered by codes and regulations-are also important. The features of these multiple hazards as applicable in the case of a storage dam (Wieland, 2012; Wieland and Fallah, 2013) are:

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[1] A large dam is one more than 15m high (above the deepest foundation level) or one between 10m and 15m high satisfying one of the following criteria: (a) more than 500m long; (b) reservoir capacity exceeding $1x10^6$ m^3; (c) spillway capacity exceeding 2000 m^3/sec.

[2] The maximum water waves in reservoirs recorded during the March 11, 2011 Tohoku earthquake in Japan (magnitude 9.0) was less than half a meter.

- Ground shaking causes vibrations and structural distortions in dams, appurtenant structures and equipment, and their foundations;
- Fault movements in the dam foundation or discontinuities in dam foundation near major faults can be activated, causing structural distortions;
- Fault displacement in the reservoir bottom may cause water waves in the reservoir or loss of freeboard;
- Rock falls and landslides may cause damage to gates, spillway piers (cracks), retaining walls (overturning), surface powerhouses (cracking and puncturing and distortions), electromechanical equipment, penstocks, masts of transmission lines, etc;
- Mass movements into the reservoir may cause impulse waves in the reservoir;
- Mass movements blocking rivers and forming landslide dams and lakes whose failure may lead to overtopping of run-of-river power plants or the inundation of powerhouses with equipment, and damage downstream;
- Ground movements and settlements due to liquefaction, densification of soil and rock fill, causing distortions in dams; and
- Abutment movements causing sliding of and distortions in the dam.

Ground shaking affects all civil structures and hydro-mechanical and electro-mechanical components of a large dam at the same time-and at times it is seen that surface structures are subjected to greater impact than the underground ones. In contrast to ground shaking the other seismic hazards listed above may only affect certain types of structures or equipment, and yet bring serious impacts. For example, the Wenchuan and Sikkim earthquakes have shown that in the mountainous epicentral region, mass movement-rock falls, landslides, rock impacts-was a major hazard, which was underestimated in the design of hydropower plants. Consequentially, construction equipment could not be transported to several dam sites for several months because access roads were blocked by rock falls. The referred cases highlight the risks associated with the assumption that a damaged dam has to remain safe and prevent any catastrophic release of stored waters for several months after an earthquake before it can be rehabilitated or transformed into a safe state. Therefore, in the earthquake design of dams all seismic hazard aspects must be considered and prioritized depending on the local conditions of a dam project.

With the increasing knowledge and safety-awareness, the dam safety concepts have undergone large scale changes, with visible impact on earthquake safety as well. In the past, dam safety was mainly related to structural safety, but today dam safety means structural safety, dam safety monitoring, operational safety and emergency planning. In terms of seismic safety, these four elements of dam safety mean the following (Wieland and Mueller, 2009):

- *Structural Safety*: strength to resist seismic forces without damage; capability to absorb high seismic forces by inelastic deformations (opening of joints and cracks in concrete dams; movements of joints in the foundation rock; inelastic deformation characteristics of embankment materials); stability (sliding and overturning stability); design of dam according to state-of-practice, etc.
- *Dam Safety Monitoring*: strong motion instrumentation of dam and foundation; visual observations and inspection after an earthquake; data analysis and interpretation; post-earthquake safety assessment, etc.
- *Operational Safety*: rule curves and operational guidelines for post-earthquake phase; experienced and qualified staff, etc.
- *Emergency Planning*: water alarm; flood mapping and evacuation plans; safe access to dam and reservoir after a strong earthquake; capability of lowering of reservoir after a strong earthquake in a controlled manner; engineering back-up, etc.

The main safety concern is the failure of a dam and the uncontrolled release of the reservoir water with flood consequences (loss of life, economical damage, environmental damage etc.), which will usually exceed the economic damage to the dam. For the seismic risk assessment of a dam, full reservoir is the critical situation that has to be analysed to assess what will be the residual strength and integrity of the dam and abutments, which will avoid any post-event after effects. Evidently, minimization of all risks associated with dam failure is the main goal of dam safety, which further translates into sub-goals of:

- Minimization of the occurrence/probability of dam failure-by ensuring structural safety, dam safety monitoring and operational safety; and
- Minimization of the consequences of dam failure-by ensuring emergency planning.

To reach above goals, a comprehensive dam safety concept is needed. Such comprehensive dam safety concepts needs to be made mandatory for large storage dams, and public dam safety agencies are also needed to enforce them. In India, substantial efforts have gone in evolving of a comprehensive dam safety concept, which is viable for the Indian conditions and generally acceptable to the dam owners and the dam operators. However, further structural and institutional level reforms will be needed for full enforcement of the comprehensive dam safety concept. The Central Dam Safety Bill-currently under consideration of the Parliament-and the ongoing Dam Rehabilitation & Improvement Project (DRIP)-with World Bank funding-are expected to give the desired impetus in this direction.

DESIGN EARTHQUAKES & GROUND MOTION DESIGN PARAMETERS

In India, the ground motion parameters for the design of large dams are arrived at by site-specific seismic design parameter studies (CWC, 2011). These study reports are generally prepared by reputed institutes working in the field of seismic sciences-e.g. Indian Institute of Technology (Roorkee), Central Water & Power Research Station (Pune) etc.-and approved by the National Committee on Seismic Design Parameters (NCSDP), which is a high level inter-disciplinary committee constituted by the Ministry of Water Resources. For the large dams falling in seismic zone III, IV or V, the approval of NCSDP on the assessment of design earthquake parameters is mandatory; and for projects in seismic zone II, the approval of NCSDP is mandatory only for dams exceeding 30 meters in height. There is no intrinsic difference in the methodology of selecting earthquake parameters for design of new dams or safety evaluation of older dams. However, the rehabilitation of existing structures which are designed on the basis of standard earthquake principles are generally not called for fresh site specific seismic studies unless new seismic activity are reported in or around the project sites.

The largest believable earthquake that can reasonably be expected to be generated by specific seismic source zone (SSZ)[3] in a given seismo-tectonic framework is referred to as Maximum Credible Earthquake (MCE)[4]. This is the largest event that could be expected to occur in the region under the presently known seismo-tectonic environment; and the structural system, if designed on this basis, would prove to be highly uneconomical. Therefore a Design Basis Earthquake (DBE), which would have a reasonable chance of occurrence during the life time[5] of the structure, is also evaluated keeping in mind the degree of safety required. The DBE represents that level of ground motion at dam site at which only minor and easily repairable damage is acceptable. Since the consequences of exceeding the DBE are mainly economic, theoretically DBE shall be determined from an economic analysis. However, from practical considerations the DBE is chosen for a certain return period of the ground motion. And, for the design of temporary structures-such as coffer dams-the Construction Earthquake is evaluated which takes into account the shorter service life of the temporary structure.

Thus, different design earthquakes (MCE, DBE and CE) are required to be evaluated and used for different components of a dam project, keeping in mind the safety criteria to be enforced for each component. The strictest safety criteria (say, safety criteria Level 1) needs to be applied for such dam elements and components, which must remain in an operable state even after a major earthquake (i.e.

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[3] The probable maximum seismic potential of the SSZ is generally controlled by the area under strain build-up, governing the length and breadth of the seismic rupture, strength and deformation characteristics of the rock, stress drop, and failure mechanism. The seismic potential is rated in terms of the magnitude of the events, and the maximum magnitude thus corresponds to the probable maximum rupture parameters.

[4] In recent periods, the terminology of Safety Evaluation Earthquake (SEE) is also being used in place of MCE. The usage of SEE has been characteristically different than MCE on account of application of a probabilistic approach wherein the choice of return period of event is linked with hazard potential categorization of dam.

[5] Most of the time, service life of the dam is expected to stretch indefinitely even beyond 100 years(the typically assumed economic life of an irrigation project) owing to ever increasing pressures on land and water due to growing populations in many countries. For example, service life of Mulla Periyar dam in India is nowhere near its end even after 150 years of operation.

ground motion of the MCE order) so that the reservoir can be lowered, and stored water can be released safely (e.g bottom outlet and spillway gates). A different safety criteria (Level 2) needs to be applied the dam body and other water retaining elements, which must be able to retain the water in the reservoir after the MCE until the water in the reservoir can be lowered if the dam experiences structural damage. A little lower criteria (Level 3) governed mainly by the economic considerations-may be applied for the appurtenant structures such as the powerhouse, switchyard, and transmission towers etc., which have to be able to withstand a design ground motion that is less severe than that of the MCE. Special criteria (Level 4) will also be needed for temporary structures and for special construction phases (e.g. open cut slopes and underground caverns in partially excavated/constructed states). For other non-critical structures at dam project sites-like office buildings, storage facilities etc.-the site specific design parameters may not be essential, and these may be designed according to building code regulations. The application of the above discussed safety criteria approach, as applied to a typical dam project, is summarized in Table 1 below:

Table 1: Design Earthquakes for Different Project Components

Project Component	**Structural Element**	**Design earthquake**		
		CE	**DBE**	**MCE**
Diversion Facilities				
Civil	Intake/outlet structure; Tunnel; Tunnel liner	X		
Geotechnical	Rock slope; underground facilities; cofferdams	X		
Electrical/Mechanical	Gates & hoisting equipments	X		
Irrigation/ Water-supply Outlets; Power Intake Structures and Power Tunnel				
Civil	Intake/outlet structure; Tunnel; Tunnel liners; Penstock steel liners; Shafts; Galleries		X	
Geotechnical	Rock slope; underground facilities; portals		X	
	Works in partially excavated/ constructed state	X		
Electrical/Mechanical	Gates & hoisting equipments; Trash racks		X	
Dam				
Dam body	Individual blocks; Crest bridge; Crest spillway piers & hoist platforms; Bottom outlet spillway piers & hoist platforms; Energy dissipation str.		X	X
Abutments/ Foundation	Abutment wedges; Foundation stability		X	X
	Foundation pit slopes including below river bed	X		
Electrical/ Mechanical	Main/Service gates & valves; operating equipment		X	X
	Guard/ Emergency gates; stop logs; operating equipments		X	
Spillway	Crest spillway; Spillway approach channel; Spillway rock excavation		X	X
	Plunge pool & overlooking slopes		X	
Powerhouse and Appurtenant Structures				
Underground powerhouse	Underground cavern; Rock slopes/ support; Access tunnels; Substructure; Superstructure		X	
	Works in partially excavated/ constructed state	X		
Surface Powerhouse	Foundation; Substructure; Superstructure		X	
Generator & excitation system	Stationary components; Rotating components		X	
Turbines	Turbine components		X	
Transformers	Transformers and related components		X	
Other Electrical/ Mechanical	Cranes & lifting devices; Gates & valves; operating equipment		X	
Switchyard	Electrical components; Masts; Transmission towers		X	

Dam Site Access				
Essential approaches for post earthquake operations	Roads; Culverts; Bridges; Road tunnels		X	
Reservoir				
Reservoir rim	Critical slopes influencing safety of dam body, or large volumes producing high waves			X
	Non-critical slopes		X	

The damage to dam and appurtenants due to earthquake is a measure of the earthquake intensity at any dam project site, and it depends on the distance and energy released from the rupture within SSZ. The Ground motion at dam project site can be characterized by peak values of expected acceleration, velocity, and/or displacement. Ideally, all factors affecting ground motion should be considered for evaluation of these parameters; but this is not practical. Generally, one source factor (magnitude) and a single transmission path factor (distance) only are considered. The local site effects are often disregarded, or limited to simple distinction between rock and alluvial sites and possible consideration of near-field effects. Empirical relations derived from available earthquake data (attenuation relations[6]) relate ground motion parameters to distance from the source and to magnitude.

The different ground motion parameters that are needed for seismic design of dam projects, and which are obtained as an outcome of site specific seismic parameter studies are as under:

- *Peak ground acceleration (PGA)*: PGA of both horizontal and vertical earthquake components are evaluated for the MCE and DBE conditions.
- *Duration of shaking*: Duration of strong shaking is an important parameter for dam safety because of its direct relation to damages, especially in case of embankments. The strong-motion duration is a function of the frequency and represents the sum total of the durations of all the strong motion segments contributing 90% of the energy of complete motion (Trifunac and Brady, 1975).
- *Response spectra*: The response spectrum represents the maximum response (in absolute acceleration and relative velocity or relative displacement) as a function of natural time period, for a given damping ratio[7], of a set of single-degree-of-freedom (SDOF) systems subjected to a time dependent excitation. Generally, the acceleration response spectra of both horizontal and vertical earthquake components are obtained from the site specific studies.
- *Acceleration time histories*: Spectrum-compatible acceleration time histories-specifying earthquake motion in time domain-are needed in case of dams with high or extreme hazard potentials, and for use of non-linear analysis techniques. Acceleration time histories may be specified for horizontal and/or vertical motion and should preferably be represented by real accelerograms obtained for site conditions similar to those present at the dam site under consideration. However, since strong ground motion data currently available do not cover the whole range of possible conditions, this is essentially an exercise in generation of random waveforms (keeping in view the duration of the ground motion and the general pattern of ground motion history) which are synthetic[8] in nature.

There are two basic approaches to developing site-specific response spectra: deterministic and probabilistic. In the deterministic approach, one or more earthquakes are specified by magnitude and location with respect to a site. Usually, the earthquake is taken as the Maximum Credible Earthquake (MCE), and assumed to occur on the portion of the source closest to the site. The site ground motions are then estimated deterministically, given the magnitude and source-to-site distance. In the probabilistic

[6] Attenuation relations are empirical relations developed from Strong Ground Motion (SGM) measurements, and are used for the prediction of expected ground motion and its intrinsic variability at the project site. Such predictions are generally performed using ground-motion models that describe the distribution of expected ground motions as a function of a few independent parameters, such as magnitude, source-to-site distance and site classification

[7] Damping values for analysis of concrete and masonry dams may be taken as 5 and 7 percent respectively when the response is assumed to be predominantly elastic. Damping values for the analysis of embankment dams may be taken as 10 to 15 percent.

[8] The artificially generated acceleration time histories of the horizontal and vertical earthquake components shall be stochastically independent; and to account for aftershocks, it is recommended to increase the duration of strong ground shaking.

approach, site ground motions are estimated for selected values of probability of ground motion exceedance in a design time period or for selected values of the annual frequency or return period of ground motion exceedance. A probabilistic ground motion assessment incorporates the frequency of occurrence of earthquakes of different magnitudes on the various seismic sources, the uncertainty of the earthquake locations on the various sources, and the ground motion attenuation including its uncertainty.

The subject matter of seismic parameter assessment still not being an exact science, and there being a number of schools of thoughts with diverse viewpoints about it, the task of finalization of seismic design parameters for different dams has not been easy for NCSDP. Figure 1 below (supported by data given in Annexure-I) indicates the spread of Peak Ground Acceleration (PGA) values approved by NCSDP for about 90 projects in recent times, underlining the challenges of discretions exercised by the Committee in arrival of these values. In order to bring some uniformity in overall approach, a guideline has also been formulated by the NCSDP for the preparation and submission of site specific seismic study report incorporating state-of-the-art provisions outlined by ICOLD, USACE etc. However, for reasons enumerated above, even formulation of the guideline has been a long drawn affair; and even after its finalization in 2011, the guidelines is being reviewed and updated from time to time.

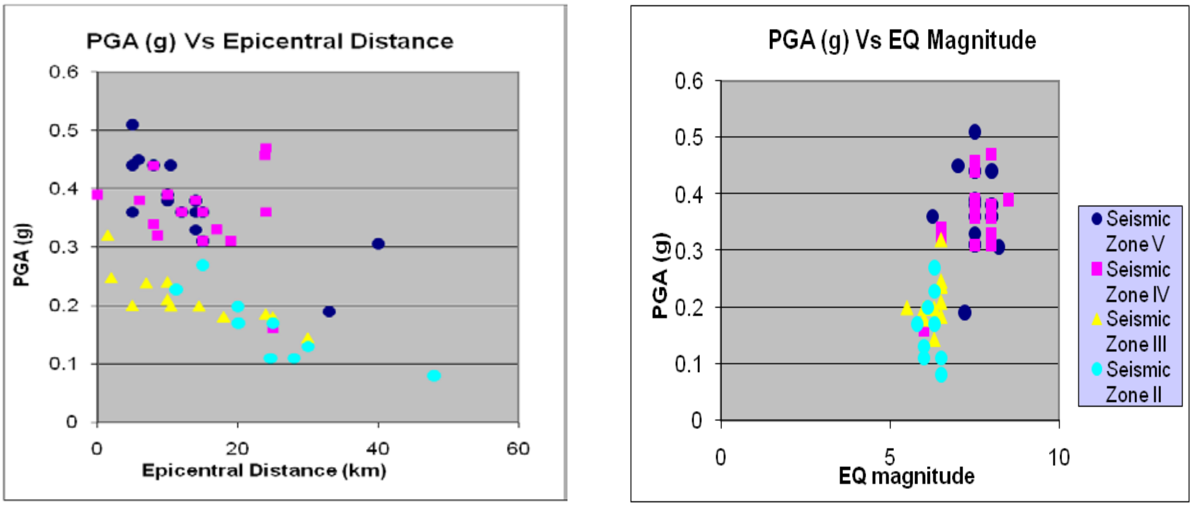

Fig. 1 Spread of PGA Values (MCE Condition) Approved by NCSDP

SEISMIC DESIGN OF DAMS

The earthquake-resistant design is only one element in the comprehensive safety of large storage dams. However, in general, dams, which can resist strong ground shaking, will perform well also under other types of static and dynamic actions. A lot of know-how exists already on the seismic behaviour of dams, but it is necessary that this information is fully used by the dam community. It is still much cheaper to make a dam to perform well during an earthquake in the design phase than having to upgrade it later. We also need to recognize that:

- A faulty design employed repeatedly in the past does not become correct when carried out in the same way the next time;
- Designs of structures to resist extreme loads may never have been tested in prototype;
- There may be unintended reserves of strength in an old design which can reduce the costs and efforts of retro fitting.

At dam sites located on active or potentially active faults or discontinuities in the dam foundation, which can be moving during a strong earthquake, only conservatively designed earth core rockfill dams should be built. This means that in highly seismically active regions where there are doubts about possible movements along discontinuities in the dam, the embankment dams are more preferable.

The only dams that are known to have failed completely as a result of seismic shaking are tailings and hydraulic fill dams, or also relatively small earthfill embankments of older and, perhaps, inadequate design and construction. Large concrete dams, which were exposed to a strong earthquake, have experienced severe cracking but none of these dams have been subjected to uncontrolled release of reservoir water. There are several design details that are regarded as contributing to the favourable seismic performance of concrete dams; and these are:

- Maintenance of low concrete placing temperatures to minimize initial heat-induced tensile stresses and shrinkage cracking
- Development and maintenance of a good drainage system.
- Providing well-prepared lift surfaces to maximize bond and tensile strength
- Utilization of shear keys in vertical construction joints
- Minimizing of discontinuities in the dam body to prevent local stress concentrations.
- Increasing the crest width to improve the dynamic stability of the dam crest
- Avoiding a break in slope on the downstream faces of gravity dams to eliminate local stress concentrations.

1. Concrete Dams

The overall process of seismic design and evaluation of concrete dams consist of the following steps:

- Development of design earthquakes and associated ground motions;
- Establishment of performance levels and performance goals;
- Analysis methodology for computation of seismic response; and
- Interpretation and evaluation of results to assess dam safety.

Earthquake ground motions for the design and evaluation of concrete hydraulic structures are the Design Basis Earthquake (DBE) and the Maximum Creditable Earthquake (MCE) ground motions. Seismic forces associated with the DBE are considered unusual loads. Those associated with the MCE are considered extreme loads. Earthquake loads are to be combined with other loads that are expected to be present during routine operations.

Three performance levels are considered for evaluation of earthquake responses of dams are shown in Figure 2 below:

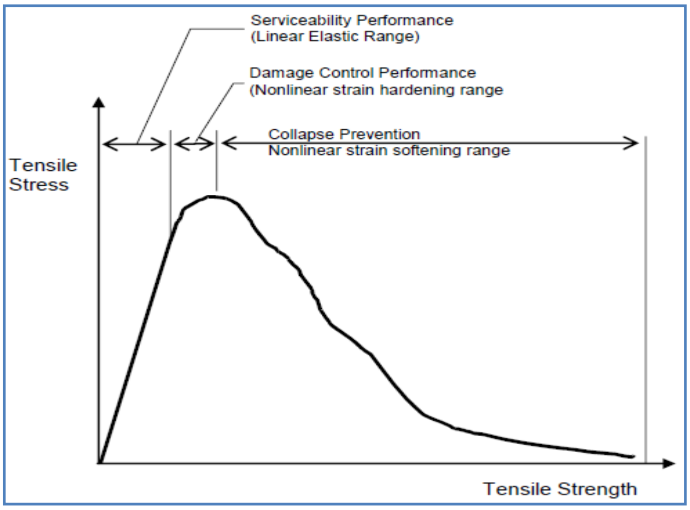

Fig. 2 Stress-Strain relationship for plain concrete structures illustrating three performance level

These performance levels for evaluation of earthquake responses of dams are further described as under:

- *Serviceability Performance*: Dam is expected to be serviceable and operable immediately following earthquakes producing ground motions up to the DBE level.
- *Damage Control Performance*: Certain parts of the dam can deform beyond their elastic limits (non-linear behaviour) if non-linear displacement demands are low and load resistance is not diminished

when the dam is subjected to extreme earthquake events. Damage may be significant, but it is generally concentrated in discrete locations where cracking and joint opening occur. At design stage all potential damage regions should be identified, and ensured that the structure is capable of resisting static loads and if necessary can be repaired to stop further damage by non-earthquake loads. Except for unlikely MCE events, it is desirable to prevent damage from occurring in substructure elements, such as foundation, and other inaccessible structural elements.

- *Collapse Prevention Performance*: Collapse prevention performance requires that the dam not collapse regardless of the level of damage. The dam may suffer un-repairable damage with nonlinear deformation greater than those associated with the damage control performance but should not result in uncontrolled release of water. Collapse prevention performance should only be permitted for unlikely MCE events. Collapse prevention analysis should be evaluated using nonlinear dynamic procedures.

1.1 Design Requirements of Concrete dams

A Strength Design

Strength design for dams subjected to earthquake ground motions is achieved by reducing the probability of structural collapse to an acceptable level. This is accomplished by selecting a representative design basis earthquake event to be used in combination with specific design and evaluation procedures that assure the structure will perform as intended. Seismic design and evaluation is most often based on linear-elastic response-spectrum or time-history analysis procedures, although nonlinear analysis procedures can be used for evaluation of certain nonlinear mechanisms. The design basis earthquake event used for strength evaluation is the Maximum Creditable Earthquake (MCE).

Live loads to be considered are those that are likely to be present during the design earthquake event. The earthquake load may involve multi-component ground motions with each component multiplied by +1 and -1 to account for the most unfavourable earthquake direction. Following is used as the Strength design loading combination:

$$QDC=QD+QL+QMCE$$

where:

- QDC=Combined action of dead, live, and MCE loads for use in evaluating damage control performance
- QD=Dead load effect
- QL=Live load effect + uplift
- QMCE=Earthquake load effect from MCE ground motions including hydrodynamic and dynamic soil pressure effects

B Serviceability Design

Serviceability design for dams subjected to earthquake ground motions is achieved by reducing the possibility of structural damage to a negligible level. As for strength performance, this is accomplished by selecting an appropriate design basis earthquake event to be used in combination with appropriate design and evaluation procedures. Evaluation is based on linear-elastic response spectrum analysis or time history analysis procedures. The design basis earthquake event used for serviceability evaluation is the Operating Basis Earthquake (DBE).

Live loads to be considered are those that are likely to be present during the DBE earthquake event. Following is used as the Serviceability loading combination:

$$QS=QD+QL+QDBE$$

where:

- QS=Combined action of dead, live, and DBE loads for use in evaluating serviceability performance
- QD=Dead load effect
- QL=Live load effect + uplift

- QDBE=Earthquake load effect from DBE ground motions including hydrodynamic and dynamic soil pressure effects

1.2 Analysis Methodology of Concrete dams

Progressive analysis methodology shall be adopted where the seismic evaluation is performed in phases in order of increasing complexity progressing from simple equivalent lateral force methods, to linear elastic response-spectrum and time-history analysis, to nonlinear methods, if necessary. For use in linear dynamic analysis of the full dam (not dam block), at least three time-histories (for each component of motion) should be used for each design earthquake.

Gravity dams with simple geometry may initially be analysed using the equivalent lateral force method, but generally 2D or 3D finite-element dynamic analysis will be required. Dynamic interactions with the foundation rock and the impounded water should be considered. Foundation rock may be idealized using simplified massless model, viscoelastic with inertia and damping effects, or a finite-element mesh with transmitting boundaries. The dam-water interaction effects may be represented by the Generalized Westergaard added-mass, an incompressible fluid mesh, or a compressible fluid mesh with energy loss capability at the reservoir bottom due to sediment accumulation.

1.3 Evaluation Procedure for Concrete dams

Evaluation of seismic performance of concrete dams starts with utilization of a demand-to-capacity ratio (DCR) as a performance indicator, then progresses to the use of performance threshold curves using both DCR and cumulative inelastic duration, and finally continues with the extent of irrecoverable movements caused by sliding and rotation, as appropriate.

The DCR for plain concrete is defined as the ratio of computed tensile stress to tensile strength of the concrete. For gravity dams, DCR is computed using principal stress (ft) demands. For this purpose, the tensile strength or capacity of the plain concrete can be obtained from the uniaxial splitting tension tests or from the static compressive strength, fc, using the relation due to (Raphel, 1984), $ft = 1.7\, fc^{2/3}$ where fc is in psi or $ft = 0.324\, fc^{2/3}$ where fc is in MPa, as recommended in USACE Manual EM 1110-2-6051. The performance threshold curves are used to assess the results of linear time-history analysis. They provide a measure of severity of the nonlinear response in terms of amount of cracking and joint opening. Finally, irrecoverable movements which are obtained by conducting nonlinear time-history analysis are used to assess stability condition of the dam under severe earthquake ground shaking.

1.4 Acceptance Criteria for Response-spectrum Analysis of Concrete dams

A linear-elastic response-spectrum analysis is generally the first step in the evaluation process. The earthquake demands in terms of principal tensile stresses are computed and compared with the tensile stress capacity of the concrete to assess whether the resulting DCR ratios meet the allowable values listed in Table 2. In cases where DCR limits for tensile stresses are exceeded, a linear-elastic time-history analysis is generally performed and evaluated, as discussed next.

Table 2: DCR Allowable Values for Response-Spectrum Analysis of Concrete Dams

Action In terms of stresses	**Performance Objectives**	
	Damage Control (MCE)	**Serviceability (DBE)**
Tension due to flexure	1.5	1.0
Diagonal tension due to shear	0.9	0.8
Shear due to sliding	1.0	0.8

1.5 Acceptance Criteria for Linear Time-history Analysis of Concrete dams

The acceptance criteria for the linear-elastic time- history analysis of concrete dams are based on the use of performance threshold curves (EM 1110-2-6051, 6053). The dam response to the MCE is considered to be within the linear-elastic range of behaviour with little or no damage if computed stress demand-capacity ratios are less than or equal to 1.0. The dam is considered to exhibit nonlinear response in the form opening and closing of contraction joints and cracking of the horizontal joints (lift lines) and the concrete if the estimated demand-capacity ratios exceed 1.0. For DCRs exceeding unity the performance is evaluated considered acceptable if demand-capacity ratios are less than 2.0 and the percent of overstressed dam-section surface areas and the cumulative duration of stress excursions above the tensile strength of the concrete fall below the performance threshold curves given in Figure 3. Consideration should also be given to relation between the fundamental period of the dam and peak of the earthquake response spectra. If lengthening of the periods of vibration due to nonlinear response behavior causes the periods to move away from the peak of the spectra, then the nonlinear response would reduce seismic loads and improve the situation by reducing stresses below the values obtained from the linear time-history analysis. When these performance conditions are not met, then a nonlinear time-history analysis would be required to estimate the damage more accurately.

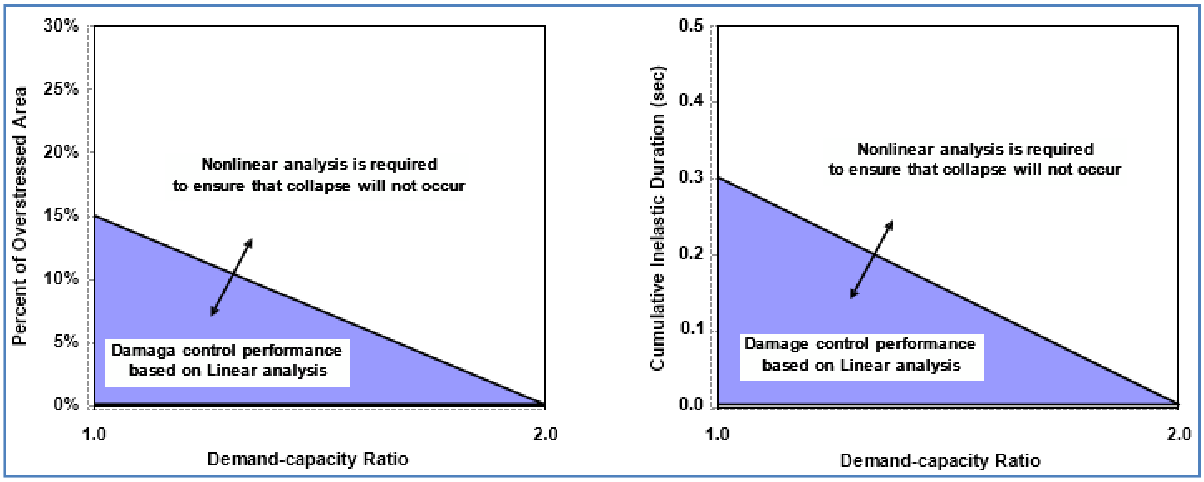

Fig. 3 Performance threshold curves for concrete gravity dams

The results of nonlinear analysis will include sliding displacement and rotation demands that must be sufficiently small not to jeopardize safety of the dam during the main event as well as during the aftershocks. For example, a linear-elastic dynamic analysis may indicate that the gravity dam will experience high tensile stresses at the dam-foundation interface and that the dam does not pass the acceptance criteria set forth for the linear analysis. In subsequent nonlinear dynamic analyses gap-friction elements are introduced at the high tensile- stress region of the base to allow formation and propagation of cracks, which may extend through the entire base of the dam. The results may indicate that the dam would fully crack leading to sliding and rocking responses with a permanent displacement (offset) at the end of the shaking. The magnitude of the permanent sliding displacement is estimated and compared with operational and safety requirements. The performance of the dam is then considered satisfactory if the cracks and permanent sliding displacement do not lead to uncontrolled release of water, and that the post-earthquake stability of the dam under static loads is not compromised.

2 Embankment Dams

One of the most dangerous consequences of the dynamic loading of an embankment dam is liquefaction of foundation or embankment zones containing cohesion-less materials of low relative density. Accordingly, the following measures are recommended during design and construction of embankment dams for improving their seismic resistance (ICOLD 2001):

- Foundation must be excavated to very dense soil or rock; alternatively the loose foundation materials must be densified or removed.
- Fill materials which tend to build up significant pore water pressures during strong shaking must not be used.
- All zones of the embankment must be thoroughly compacted to prevent excessive settlement during an earthquake.
- All embankment dams, and especially homogeneous dams, must have high capacity internal drainage zones to intercept seepage from any transverse cracking caused by earthquakes, and to assure that

embankment zones designed to be unsaturated remain so after any event that may have led to cracking.

- Filters must be provided on fractured rock to preclude piping of embankment materials into the foundation.
- Wide filter and drain zones must be used.
- The upstream and/or downstream filter and transition zones should be self-healing, and of such gradation as to also heal cracking within the core.
- Sufficient freeboard should be provided in order to cover the settlement likely to occur during the earthquake and possible water waves in the reservoir due to mass movements, etc.
- Since cracking of the crest is possible, the crest width should be wider than normal to produce longer seepage paths through any transverse cracks that may develop during earthquakes.

The dynamic response of an embankment dam during strong ground shaking is governed by the deformational characteristics of the different soil materials. For large dams, the earthquake induced permanent deformations must be calculated. The calculations of the permanent settlement of large rockfill dams based on dynamic analyses are still very approximate because of data limitations. Poorly compacted rockfill may settle significantly during strong ground shaking but may well withstand strong earthquakes. To get information on the dynamic material properties, dynamic direct shear or triaxial tests with large samples are needed. These tests are too costly for most rockfill dams. But as information on the dynamic behaviour of rockfill published in the literature is also scarce, the settlement prediction involves sensitivity analyses and engineering judgment.

Knowledge of the constraints in the dam body and of the permanent deformations are essential to the assessment of the earthquake behaviour of an embankment dam. The Figure 4 below schematically presents the commonly encountered issues of: crest settlement (with resulting loss of freeboard); longitudinal cracks (associated with large lateral oscillations); transverse cracks (associated with large longitudinal oscillations or transverse asynchrone excitations); and cracks within the dam body (piping initiation). Any of the above mentioned cracks can lead to erosion of an embankment dam.

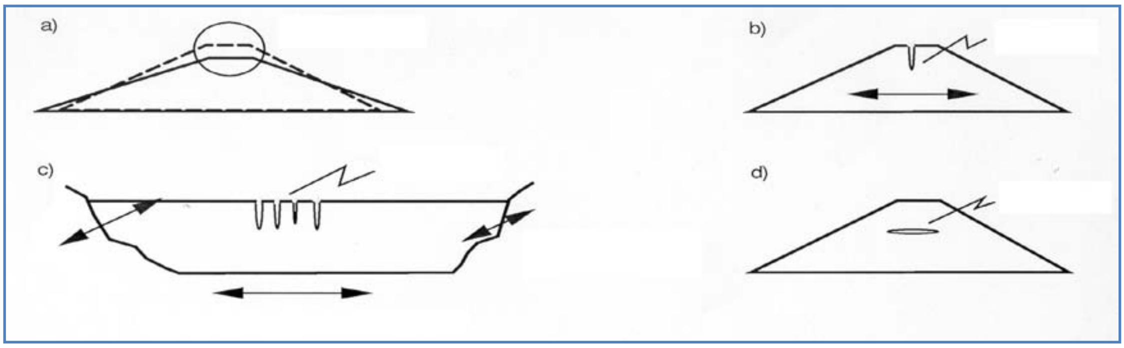

Fig. 4 Permanent deformation and cracks in an embankment dam

3 Concrete-face Rockfill Dams

The design of concrete face rockfill (CFR) dams for earthquake is mainly based on experience and engineering judgment. Methods used to estimate dam deformation induced by earthquake range from simple analytical tools to three dimensional numerical models. Analytical tools are simple to use but they cannot take into account special features of dam design, like zonification, berms or non-uniform slopes. However, the reliability of numerical methods heavily depends on the choice of the constitutive models and the selection of input parameters. Furthermore, numerical models are too involved to be used at design stage.

Many engineers have argued that the concrete face rockfill dam is inherently safe against potential seismic damage (Sherard and Cooke, 1987 a, b). This is because earthquake cannot cause pore water pressure build up and strength degradation of the free-draining compacted rockfill. Earthquake analyses in the past have focussed mainly on settlement predictions of the crest and deformations of the dam body and the stability of the downstream slopes. The numerical study of the seismic performance of modern concrete face rockfill dams carried-out by USBR (US Bureau of Reclamation) shows that crest settlements would not exceed 1 to 2% of the dam height under severe earthquake shaking.

However, the deformational behaviour of the concrete slab, which acts as a rigid diaphragm for vibrations in cross-canyon direction, is very different from that of the rockfill and transition zone material, the cross-canyon response of the rockfill may be restrained by the relatively rigid concrete slab. This may result in high in-plane stresses in the concrete slab that may be sufficiently large to cause local buckling, shearing off of the slab along the joints or to damage the plinth. It is, therefore, necessary to look carefully into the behaviour of the concrete face under the cross-canyon component of the earthquake ground shaking.

SEISMIC SAFETY OF EXISTING DAMS

The seismic safety aspects of existing dams is an important issue as most dam codes, regulations, recommendations and guidelines are primarily concerned with the design of new dams. The design of a dam, which was considered as safe at the time it was commissioned may not be safe forever. As earthquake engineering is still a relatively young discipline, design criteria, methods of analysis, design concepts etc. may be subject to changes especially when a large dam, designed according to the current state-of-practice, gets damaged during an earthquake. Thus there is a need for periodic checks of the seismic design criteria and the earthquake safety of existing large dams. Two cases, which may mandatory call for the safety evaluation of existing dams (Wieland, 2006) are:

- When a strong earthquake has occurred and strong motion instruments have recorded strong shaking in a dam and a post-earthquake inspection has revealed some damage, and
- When the seismic design criteria or seismic performance criteria have changed and/or new developments have taken place (a) in the seismic hazard assessment, (b) in the methods of seismic analysis, or (c) in the dynamic behaviour of materials, etc.
- The seismic safety of an existing embankment dam subjected to severe ground motion may not be easy to fathom; and hence their safety needs to be assessed by investigating the following:
- Permanent deformations experienced during and after an earthquake (e.g. loss of freeboard);
- Stability of slopes during and after the earthquake, and dynamic slope movements;
- Build-up of excess pore water pressures in embankment and foundation materials (soil liquefaction);
- Damage to filter, drainage and transition layers;
- Damage to waterproofing elements in dam and foundation (core, upstream concrete or asphalt membranes, geo-textiles, grout curtain, diaphragm walls in foundation, etc.);
- Vulnerability of dam to internal erosion after formation of cracks and limited sliding movements of embankment slopes, or formation of loose material zones due to high shear, etc.

Hsinfengkiang buttress dam (1962 earthquake in China, 105 m high), Koyna gravity dam (1967 earthquake in India, 103 m high), Pacoima arch dam (1971 and 1994 earthquakes in California, 116 m high), Rapel arch dam (1985 earthquake in Chile, 110 m high) and Sefid Rud buttress dam (1990 earthquake in Iran, 106 m high) are the highest concrete dams, which have been exposed to very strong ground shaking and have suffered different degrees of damage but none of them has failed. Major repair and strengthening works were carried out for the Hsinfengkiang, Koyna and Sefid Rud dams and all dams are in operation. These dams, as most of the existing dams, were designed against earthquakes, using seismic design criteria and/or methods of seismic analysis, which are considered as obsolete or incorrect today. They have also experienced ground motions that were much more severe than those expected at the time of construction. Generally, it has been found that dams that have been designed properly to resist static loads prove to also have significant inherent resistance to earthquake action. As such, there is no need to have a panic reaction on the issue of existing dams not meeting present design criteria.

1 Material Characterisation and Challenges in Seismic Safety Assessment of Existing Dams

Apart from the problems of evaluating a fresh seismic hazard for the existing dams, the materials used in the dam and their present behaviour also pose a considerable challenge to the engineers. There are no clear cut guidelines for assessing the dynamic properties of the old and ageing materials used in the dam.

One of the biggest challenges is to assess the tensile strength of the rigid dam materials and accordingly to assess the tensile stresses that can be considered permissible in the event of an earthquake.

In our country (embarked on a dam building programme from 1950 onwards), keeping in view the shortages of cement and abundant availability of manual labour, a large number of Random Rubble masonry dam ware built right up to 1980s. The world over such constructions technologies and materials have not been adopted widely; and, consequently the large body of experience exists for old concrete dams but the same cannot be directly translated to Random Rubble masonry in view of the differences in particle size distribution. During 1987, a large swarm of tremors in Bhatsa area raised serious questions on the tensile strength of partially constructed masonry dams. However, in the absence of large scale tests the judgment had to rely on qualitative assessments only.

Similarly, a number of dams were strengthened in the part by providing masonry buttresses or earth backing to improve their stability in static conditions. In many of retrofitting cases, such measures were considered adequate even for improving the stability under seismic events as well since pseudo static analysis and design methods indicated such improvements. However, the rigorous analysis of such strengthened structures may indicate otherwise or even adverse effects of such strengthening measures. It is, therefore, necessary for the dam safety assessors and managers to obtain such knowledge and standardise the measures which can help ameliorate the situation.

CONCLUSIONS

The seismic hazard is a multi-hazard for most dam projects, with ground shaking considered as the main hazard in most of the earthquake guidelines for dams. The technology for designing and building dams and appurtenant structures that can safely resist the effects of strong ground shaking is generally available, while new safety concepts are still needed for very large dams in highly seismic regions, for new types of dams–such as CFR dams, and for existing dams needing retrofitting for seismic safety. Though many small dams have suffered damage during strong earthquakes, but no large dam has failed due to earthquake shaking. Dynamic analysis of dams, either using response spectra or time history, has now become almost mandatory for all large dams being designed or planned in India. The National Committee of Seismic Design Parameters (NCSDP) of India is periodically reviewing its guidelines on seismic parameters to be adopted for dam design based on the state-of-the-art inputs. The changes in the seismic design criteria and the design concepts make it necessary to perform several seismic safety checks during the long economical life of a large dam–however, there is no reason for panic.

There is a need to have standardised design and analysis tools for helping the design community as hazard depiction and analysis methodologies are closely linked and currently there is disconnect between the two. Design aids may have to be evolved for evolving a dam conforming to the prescribed seismic hazard with a view to bring seismic design in mainstream design practice. Also, as can be seen in seismic safety of existing dams, the problems become highly non-linear due to duplex complexities involved i.e. those arising from materials and also from the geometry and interferes between two heterogeneous materials like earth and masonry. The Indian dam engineers and the seismic analysts are therefore called for to address these challenges sooner than later.

Annexure–I

PGA (MCE Condition) values for some of the NCSDP approved Projects

S. No.	Name of Project	Dam ht (m)	Seismic Zone	Magnitude	Epicentral Distance (Km)	PGA (MCE) (g)
1	Lower Subansiri	133	V	7.5	10	0.38
2	Siang Middle (Siyom) Project	154	V	7	5.8	0.45
3	Kameng (Baishom)	75	V	7.5	15	0.31
4	Kameng (Tenga)	27	V	7.5	15	0.31
5	Dikrong	61	V	7.5	14	0.33
6	Lower Siang	85	V	7.5	12	0.36
7	Panan	101	V	8	14	0.36

8	Nyamjhang	11.2	V	8	15	0.36
9	Sissiri	145.5	V	8	15	0.36
10	Hirong	133.5	V	7.5	12	0.36
11	Demwe Lower	163.02	V	8	14	0.38
12	Tato-II	155	V	7.5	5	0.44
13	Kalpong	31	V	7.5	10	0.39
14	Dhankari	32.25	V	7.5	5	0.51
15	Pagladiya	28.75	V	8.2	40	0.306
16	Parbati H.E. P Stage-II	91	V	8	15	0.36
17	Parbati H.E Project Stage-III	43	V	8	15	0.36
18	Chamera Stage III	68	V	7.5	15	0.31
19	Kol dam	163	V	7.2	33	0.19
20	Sainj HEP	24.5	V	7.5	15	0.31
21	Kutehr	27	V	7.5	15	0.31
22	Bajoli Holi	66	V	7.5	15	0.31
23	Myntdu Leska H.E. Project	59	V	8	10.4	0.44
24	Tuirial H.E.Project	77	V	6.25	5	0.36
25	Loharinag Pala H.E. Project	15	V	8	14	0.38
26	Lata Tapovan H.E. Project	16	V	8	14	0.38
27	Sewa HEP Stage-II	53	V	7.5	8	0.44
28	Indira Sagar (Polavaram H.E. P)	33	IV	6	25	0.16
29	Dagmara	11	IV	8.5	0	0.39
30	Karcham Wangtoo	98	IV	8	14	0.38
31	Malana H.E. Project Stage-II	51	IV	8	15	0.36
32	Budhil	61.5	IV	7.5	15	0.31
33	Kishanganga H.E. Project	101	IV	6.5	8	0.34
34	URI-II H.E.Project	52	IV	7.5	10	0.39
35	Nimoo Bazgo HEP	57	IV	7.5	6	0.38
36	Chutak H.E.P	15	IV	7.5	12	0.36
37	Pakal Dul (Drangdhuran) HEP	167	IV	7.5	15	0.31
38	Kiru HEP	140	IV	8	19	0.31
39	Shahpukandi	54.5	IV	7.5	15	0.31
40	Teesta Stage-V	96	IV	6.5	8.6	0.32
41	Rangit H.E. Project Stage IV	44	IV	7.5	23.9	0.457
42	Teesta H.E. Project Stage III	60	IV	8	15	0.36
43	Teesta H.E.Project Stage-VI	23.5	IV	7.5	14	0.38
44	Koteshwar	97.5	IV	7.5	15	0.31
45	Tapovan Vishnugad HEP	25	IV	8	14	0.38
46	Kotlibhel HEP-1A	82.5	IV	7.5	15	0.31
47	Kotlibhel HEP-1B	70.5	IV	7.5	15	0.31
48	Kotlibhel HEP-Stage-II	58.6	IV	7.5	15	0.31
49	Pala Maneri H.E. Project	74	IV	8	24	0.36
50	Vishnugad Pippalkoti HEP	45	IV	8	14	0.38
51	Singoli Bhatwari H.E. P	22	IV	8	14	0.38
52	Vyasi. H.E. P	86	IV	7.5	12	0.36
53	Alaknanda	20	IV	8	15	0.36
54	Shrinagar H.E.Project	90	IV	7.5	15	0.31
55	Bhairon Ghati H.E. Project	22	IV	8	15	0.36
56	Jamrani Multi Purpose Project	130	IV	7.5	15	0.31
57	Phata Byung HEP	26	IV	8	15	0.36
58	Jelam Tanak HEP	28	IV	8	15	0.36
59	Tiuni-Plasu HEP	36.3	IV	8	24	0.47

60	Teesta low dam Stage-III	29.5	IV	8	17	0.33
61	Teesta low dams Stage-IV	45	IV	7.5	8	0.44
62	Rammam-H.E. ProjectStage-III	20	IV	8	14	0.38
63	Tail Pond Dam	29.5	III	6.5	24	0.185
64	K.L. Rao (Pulichinthala) Sagar	42.24	III	6	25	0.18
65	Middle Vaitharna	102	III	6.5	10	0.21
66	Bhugad Dam	68.63	III	6.3	18	0.18
67	Khargihill dam	75.62	III	6.3	18	0.18
68	Mankulam H.E. Project	50	III	6	10.5	0.199
69	Indira (Narmada) Sagar Dam	92	III	6.5	10	0.24
70	Upper Beda Project	23.93	III	6.5	7	0.238
71	Lower Goi Project	43.8	III	6.5	2	0.247
72	Omkareshwar H.E. Project	73.12	III	5.5	5	0.2
73	Upper Narmada Project	30.64	III	6.3	30	0.144
74	Pench Diversion Project	41	III	6.3	14.5	0.199
75	Rihand	91	III	6.5	1.5	0.32
76	Jheri Dam	36.5	II	6.3	20.1	0.17
77	Mohankavchali dam	70.6	II	6.3	20.1	0.17
78	Paikhed dam	90.09	II	6.3	20.1	0.17
79	Chasmandava dam	35.4	II	5.8	25	0.17
80	Chikkar dam	29.9	II	5.8	25	0.17
81	Dabdar dam	62.4	II	5.8	25	0.17
82	Kelvan	62.4	II	5.8	25	0.17
83	Tungabhadra Dam	49.38	II	6.3	15	0.27
84	Gundia HEP	87	II	6.3	11.2	0.228
85	Kutni Feeder Reservoir Project	30.36	II	6	30	0.13
86	Kelo Project (Masonry/Concrete)	16	II	6.1	20	0.199
87	Kelo Project (earthen)	24.22	II	6.1	20	0.199
88	Makodia dam	27	II	6.5	48	0.08
89	Lower Orr Dam Project	41.84	II	6	24.7	0.11
90	Daudhan dam	77	II	6.5	28	0.11

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PRELIMINARY INVESTIGATION FOR SCREENING OF LIQUEFIABLE AREAS IN HARYANA STATE, INDIA

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ABSTRACT

Liquefaction susceptibility of state of Haryana has been analysed by using geological and geomorphological characteristics of the area. Soil resource maps, geomorphological maps, earthquake hazard maps and ground water table data have been used for assessing liquefaction potential. A liquefaction susceptibility map has been prepared which can be used as initial rough screening guide for subsequent detailed assessment of liquefaction vulnerability of the state. The state has been classified into three zones in terms of liquefaction susceptibility value (L_s) for its proneness to liquefaction; high $(L_s \geq 0.8)$, moderate $(0.5 < L_s < 0.8)$ and low $(L_s \leq 0.5)$. It has been observed that the high susceptible areas lie mainly in National Capital Region (NCR) and extend along the National Highway-1 (NH-1). The results have also been validated using semi-empirical procedure based on geotechnical criteria.

KEYWORDS: Earthquake, Geology, Geomorphology, Liquefaction susceptibility

LIST OF SYMBOLS

L_s-Liquefaction Susceptibility Value

$PGA\ (g)$-Peak Ground Acceleration

V_s-Shear Wave Velocity

SPT-Standard Penetration Test

INTRODUCTION

Liquefaction and related phenomena have been responsible for heavy damages in past earthquakes round the world particularly in urban areas. Liquefaction related issues evolved in India in wake of Bihar (1934) and Bhuj (2001) earthquakes. Different liquefaction features of sand boils, craters, lateral spreading etc. were observed during these earthquakes (Rajendran et al., 2001). Soil liquefaction has been a major cause of damage to life and property in these earthquakes and it clearly poses a significant threat to life and property in other states too during future earthquakes.

Based on the type of data available, liquefaction hazard mapping can be carried out by different methods, e.g. deterministic approach, probabilistic approach and susceptibility mapping based on geological and geomorphological characteristics. Iwasaki et al. (1982) and Wakamatsu (1992) have correlated liquefaction susceptibility to geomorphological and geological characteristics. Similar methodology was suggested for Shonai Plain, Japan (Kotoda et al., 1988). During the assessment of liquefaction susceptibility, the age of deposit and depth of water table are also considered important factors (Obermeier, 1996). Manmade fills and young Holocene sediments in particular are susceptible to liquefaction (Youd and Perkins, 1978). Similar studies have been conducted for Chennai City, India (Ganapathy and Rajawat, 2012), Laoag City, Northern Philippines (Beroya and Aydin, 2007) and Delhi, India (Mohanty et al., 2007).

In the present study, liquefaction susceptibility of state of Haryana has been analysed by using geological and geomorphological characteristics of the area. The Grade-I hazard map (TCEGE, 1999) developed in the present study would serve as a rough guide for identifying zones where earthquake induced liquefaction is anticipated and hence a detailed investigation may be required.

DESCRIPTION OF THE STUDY REGION

Haryana is the Northern State of India, sprawling over an area of 44212 km^2. It ranks 19^{th} in terms of area in the country. It is surrounded by the states of Uttarakhand, Himachal Pradesh and Shiwalik hills on the North, Uttar Pradesh on the East, Punjab on the West and Delhi, Rajasthan and Aravali hills on the South. It is positioned between $27^o\ 37^{'}$-$37^o\ 35^{'}$ latitude and between $74^o\ 28^{'}$-$77^o\ 36^{'}$ longitude. Its altitude ranges from 700 to 900 ft above mean sea level. Haryana has a population of 25.353 million as per census of 2011 (Govt. of Haryana, 2016).

1. Geological-cum-Geomorphological Setting

The state of Haryana and the adjoining areas are covered to a large extent by Quaternary sediments of alluvial/aeolian origin. The geological set-up of the area comprises the sub-Himalayan system of rocks, mostly belonging to Siwalik Group which is exposed in the north-eastern extremity and adjoining parts. In the south and south-western corner of Haryana bordering the state of Rajasthan, older rocks belonging to Delhi Supergroup are exposed. In between lays the vast stretch of Quaternary sediments of alluvial/aeolian origin. The different geomorphic units recognised include:(1) High structural hills, (2) Moderate structural cum denudational hills, (3) Low structural-cum-denudational hills, (4) Older and younger piedmont zones, (5) Flood plain, (6) Older Alluvial surface, (7) Aeolian zone, (8) Transitional zone and (9) Upland tract.

Except the river Yamuna flowing along the eastern boundary of the state, the only other stream is the Ghagghar. This river appears to be structure controlled and flows along well-defined tectonic lines. The southerly to south-easterly direction of flow of the river Yamuna indicates a basement high. The topographical low passing through Delhi-Rohtak-Hisar and Sirsa appears to coincide with basement high and the gradual shift in the drainage system indicates some neotectonic activity in the region (GSI, 2012).

According to assessment of Ministry of Water Resources, flood prone area in Haryana is about 23500 km2. In recent history, devastating floods hit Haryana in 1977, 1978, 1980, 1983, 1988, 1993, 1995 and 2010. The floods in Haryana occur frequently because of its physiographic situation. In Haryana, a depression saucer shape zone exists around Delhi-Rohtak-Hisar-Sirsa axis and it has a poor drainage system and sometimes heavy precipitation becomes a major contributing factor in causing floods as it was during Rohtak flood (August, 1995). The flood in these areas occurs mainly due to heavy runoff from the hilly terrain and overflow of river Yamuna in the plain areas during Monsoons (DTCP, 2010). A map prepared by Building Materials & Technology Promotion Council (BMTPC, 2007) and printed in Vulnerability Atlas of India (First Revision) is shown in Figure 1 highlighting the flood prone areas in Haryana.

2. Tectonic Setting

The State of Haryana falls in three Seismic Zones viz. II, III and IV, creating low to moderate damage risk from earthquakes. Ambala, Sonipat, Rohtak, Karnal, Gurgaon, Faridabad, Panipat, Rewari and Yamunanagar districts lie in Zone IV. The districts of Kurukshetra, Jind, Hisar, Bhiwani, Mahendragarh and Kaithal lie in Zone III, while only Sirsa District lies in Zone II (BIS, 2002). An earthquake hazard map for Haryana state is prepared by Building Materials & Technology Promotion Council (BMTPC, 2007) and printed in Vulnerability Atlas of India (First Revision) is shown in Figure 2. The region remains susceptible to earthquakes due to the following faults (Puri and Jain, 2015):

a) Aravali-Delhi Fold Belt: It includes Mahendragarh Dehradun Subsurface Fault, Mathura Fault and several major and minor lineaments.
b) Himalayan Thrust System: It includes mainly Main Boundary Thrust, Main Crustal Thrust and Jwala Mukhi Thrust along various other tectonic features.
c) Moradabad Fault.
d) Sardar Shahar Fault.

In the recent past, no major earthquakes have hit Haryana but shocks are felt whenever an earthquake occurs in areas of Himalayan Thrust System.

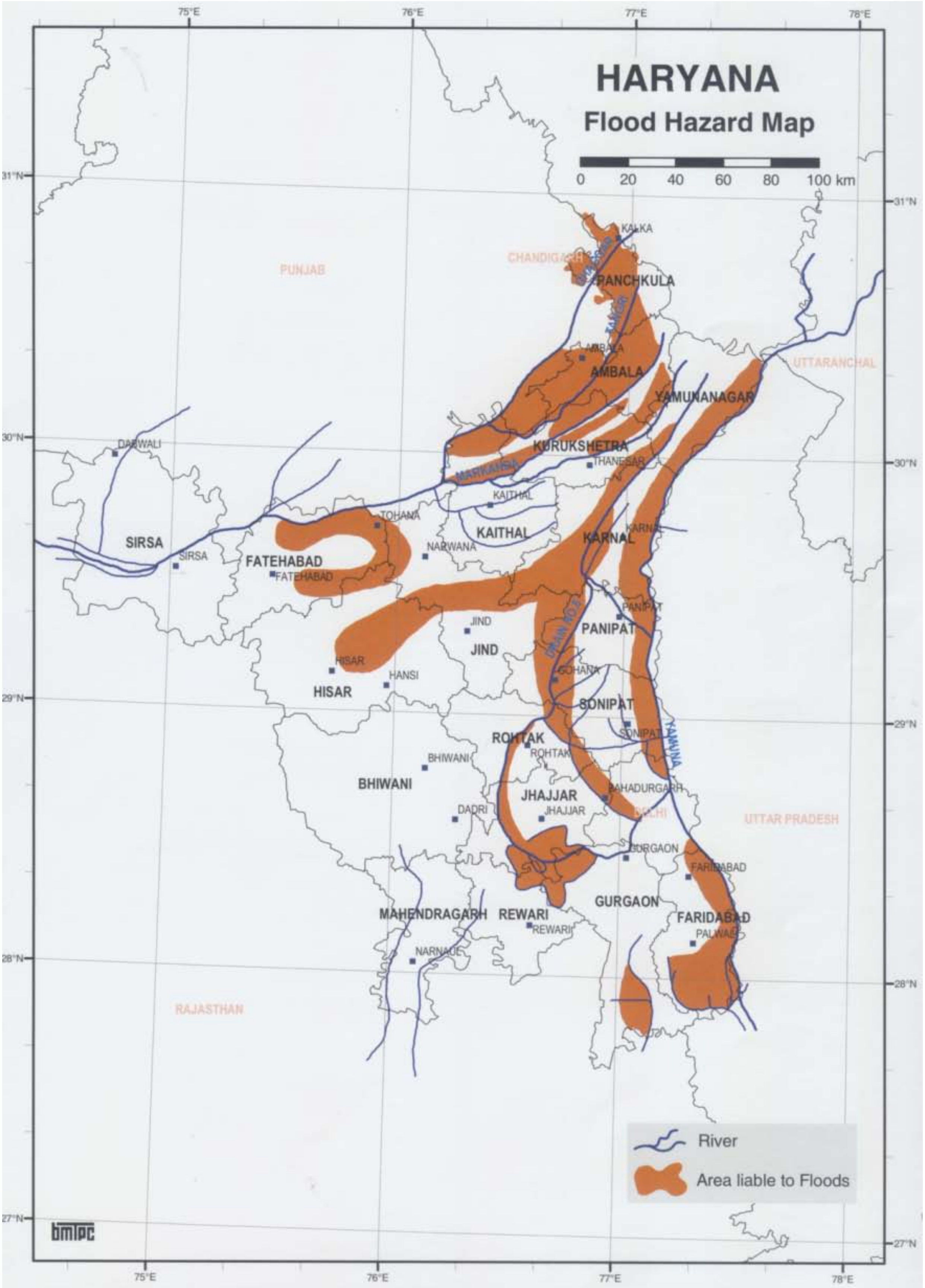

Fig. 1 Flood Hazard Map (BMTPC, 2007)

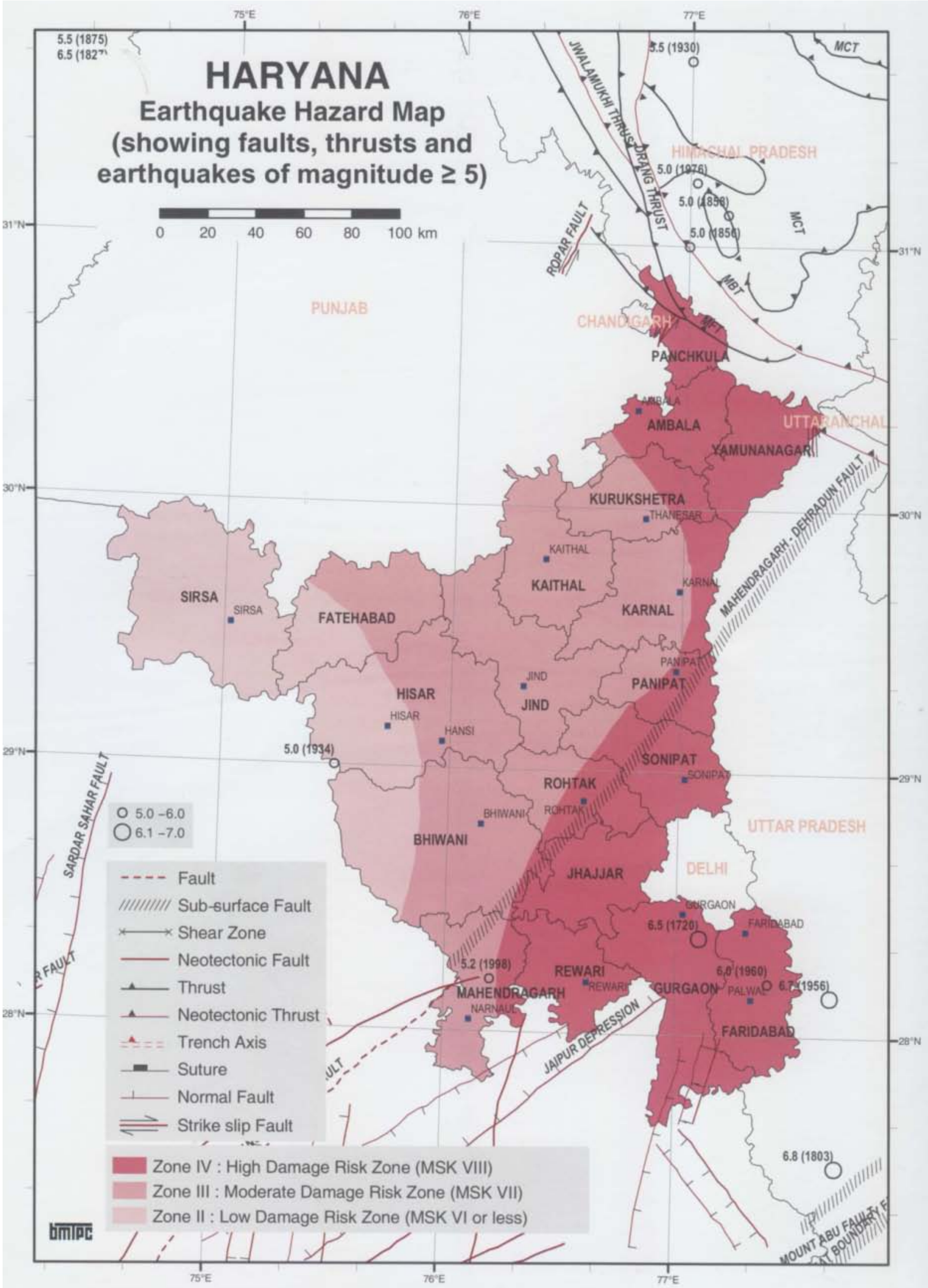

Fig. 2 Earthquake Hazard Map (BMTPC, 2007)

METHODOLOGY

Regional study based on geological and geomorphological data has been conducted to delineate areas where liquefaction could be triggered by a sufficiently large earthquake. Soil resource maps, geomorphological maps, earthquake hazard maps and ground water table data have been used for assessing liquefaction potential. A liquefaction susceptibility map has been prepared which can be used as initial rough screening guide for subsequent detailed assessment of liquefaction vulnerability of the state.

The results have also been validated using semi-empirical procedure based on geotechnical criteria (Idriss and Boulanger, 2006).

Several investigators have successfully correlated geological and geomorphological characteristics with assessment of liquefaction susceptibility. The classifications proposed by Youd and Perkins (1978), Iwasaki et al. (1982), Wakamatsu (1992) and Obermeier (1996) are reported in Tables 1 to 4.

Table 1: Liquefaction Susceptibility of Geomorphological Units (Youd and Perkins, 1978)

Type of deposits	**General distribution of deposits**	**Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by the age of deposits)**			
		<500 years	**Holocene**	**Pleistocene**	**Pre-Pleistocene**
River channel	Locally variable	Very High	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	-	Low	Very low	Very low
Marine terrace and plain	Widespread	Moderate	Low	Low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Very low
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	-	-
Residual soils	Rare	Low	Low	Very low	Very low
Sebkha	Locally variable	High	Moderate	Low	Very low
Delta	Widespread	Very High	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach- High wave energy	Widespread	Moderate	Low	Very low	Very low
Beach -Low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
Loose fill	Variable	Very High	-	-	-
Compacted fill	Variable	Low	-	-	-

Table 2: Liquefaction Susceptibility of Various Geomorphological Units (Iwasaki et al., 1982)

Rank	**Geomorphological units**	**Liquefaction potential**
A	Present river bed, old river bed, swamp, reclaimed land and inter-dune low	Liquefaction likely
B	Fan, natural levee, sand dune, flood plain, beach and other plains	Liquefaction possible
C	Terrace, hill and mountain	Liquefaction not likely

Table 3: Liquefaction Susceptibility of Various Geomorphological Units at Ground Motion of the MMS VIII (Wakamatsu, 1992)

Classification	Specific conditions	Liquefaction potential
Valley plain	Consisting of gravel or cobble	Not likely
	Consisting of sandy soil	Possible
Alluvial fan	Vertical gradient>0.5%	Not likely
	Vertical gradient<0.5%	Possible
Natural levee	Top of natural levee	Possible
	Edge of natural levee	Likely
Back marsh		Possible
Abandoned river channel		Likely
Former pond		Likely
March and swamp		Possible
Dry river bed	Consisting of gravel	Not likely
	Consisting of sandy soil	Likely
Delta		Possible
Bar	Sand bar	Possible
	Gravel bar	Not likely
Sand dune	Top of dune	Not likely
	Lower slope of dune	Likely
Beach	Beach	Not Likely
	Artificial beach	Likely
Inter-levee lowland		Likely
Reclaimed land by drainage		Possible
Reclaimed land		Likely
Spring		Likely
Fill	Fill on boundary zone between sand and low land	Likely
	Fill adjoining cliff	Likely
	Fill on marsh or swamp	Likely
	Fill on reclaimed land by drainage	Likely
	Other type of fill	Possible

Table 4: Liquefaction Susceptibility of Various Geomorphological Units (Obermeier, 1996)

Age of Deposit	Depth of Water Table		
	0-3 m	3-10 m	10 m
Latest Holocene	High	Low	Nil
Earlier Holocene	Moderate	Low	Nil
Late Pleistocene	Low	Nil	Nil

Table 5: Critical Units of Study Region

Lithology	Geomorphology	Water Table	Anticipated PGA (g) as per IS:1893 (BIS 2002)	Liquefaction
Sands and Non-Plastic Silts	Flood plains, River beds, Young deposits (age<500 years)	0 to 10 m	0.24, corresponding to zone IV	Likely
Loams	Holocene deposits	10 to 20 m	0.16, corresponding to zone III	Possible
Clays and Plastic Silts	Pleistocene and older deposits	>20 m	0.10, corresponding to zone II	Not likely

On the basis of soil resource maps, geomorphological maps, earthquake hazard maps, flood hazard maps and ground water table data of the area as reported in Figures 1 to 4 and using Tables 1 to 4, various geological, geomorphological and seismic units in the study region have been identified and have been reported in Table 5. These units have been considered in order to prepare Grade-I liquefaction susceptibility map for the State.

LIQUEFACTION ASSESSMENT

1. Lithology

Lithological characteristics like grain size and depth of soil deposits play a crucial role in determining the magnitude of ground shaking. It is because in granular soils and artificial fills (loose), the shear wave velocity (V_S) is very low and subsequent ground shaking is very high. Moreover, ground shaking during an earthquake is further amplified by the granular soils. Soils formed by processes that lead to a uniform grain size distribution and deposition in loose state are likely to liquefy when saturated (Sitharam et al., 2004).

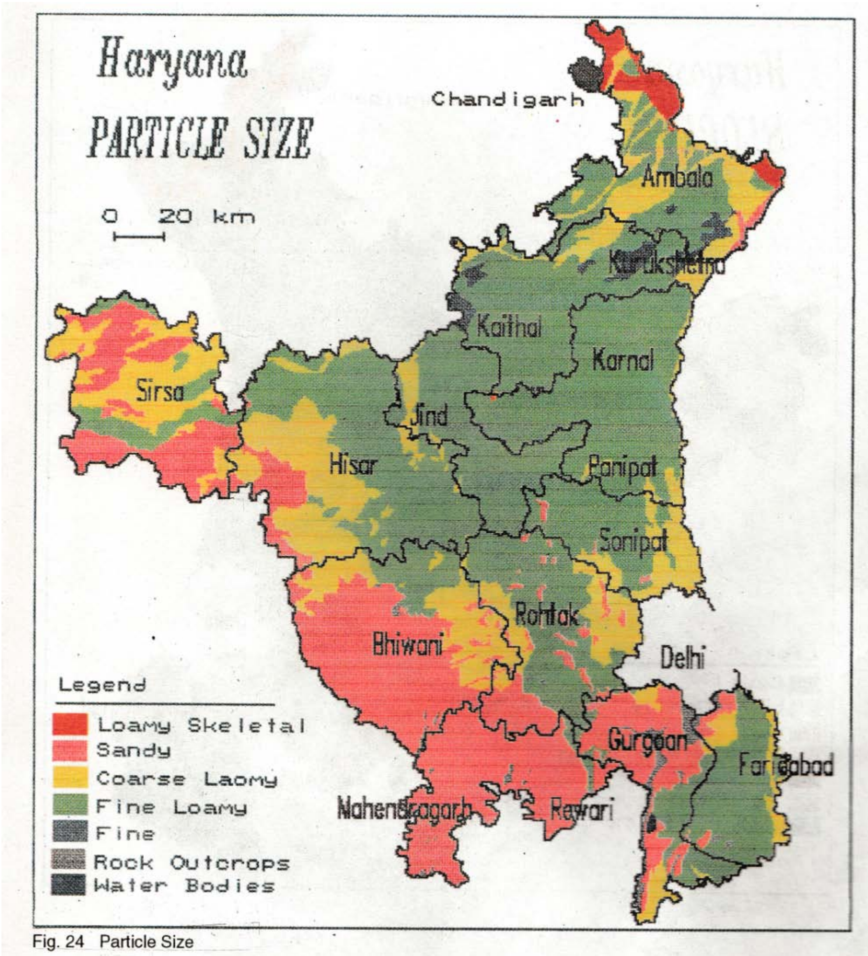

Fig. 3 Soil Resource Map (Sachdev et al., 1995)

A soil resource map has been developed by National Bureau of Soil Survey (NBSS) and Land Use Planning (Sachdev et al., 1995) at a scale of 1:250000 has been used in the study and is shown in Figure 3. It has been observed that in districts of Bhiwani, Gurgaon, Mahendragarh, Rewari and some parts of Sirsa and Hisar, sand is the major soil type. Hence, these districts are highly susceptible to liquefaction during earthquakes. In districts of Ambala, Kaithal, Kurukshetra and Mewat, large areas with fine grained soils have been observed, which makes these regions less susceptible to liquefaction. In most parts of Panchkula District, rock layer has been observed at surface or at shallow depth, which makes the region not or less susceptible to liquefaction hazard.

In other regions, loamy soils are found in abundance, which makes them moderately susceptible to liquefaction. However, for loamy soils, strong experimental basis is required to conclude whether they would liquefy or not during earthquakes (Boulanger and Idriss, 2006).

2. Geomorphology

Flood plains are expected to have sand deposits underneath silt-clay layers (Beroya and Aydin, 2007). Hence these floods plains are susceptible to liquefaction during earthquakes. A large part of Haryana along National Highway-1 comes under the category of flood plains as shown in Figure 1. Another important geomorphological unit is age of the deposit. Young deposits are more susceptible to liquefaction as compared to the older deposits. A map describing the age of soil deposits of Haryana prepared by Geological Survey of India (GSI, 1973) is shown in Figure 4. It has been observed that soil deposits in Haryana belong to Holocene and Pleistocene age group. Hence their susceptibility to liquefaction is moderate to high.

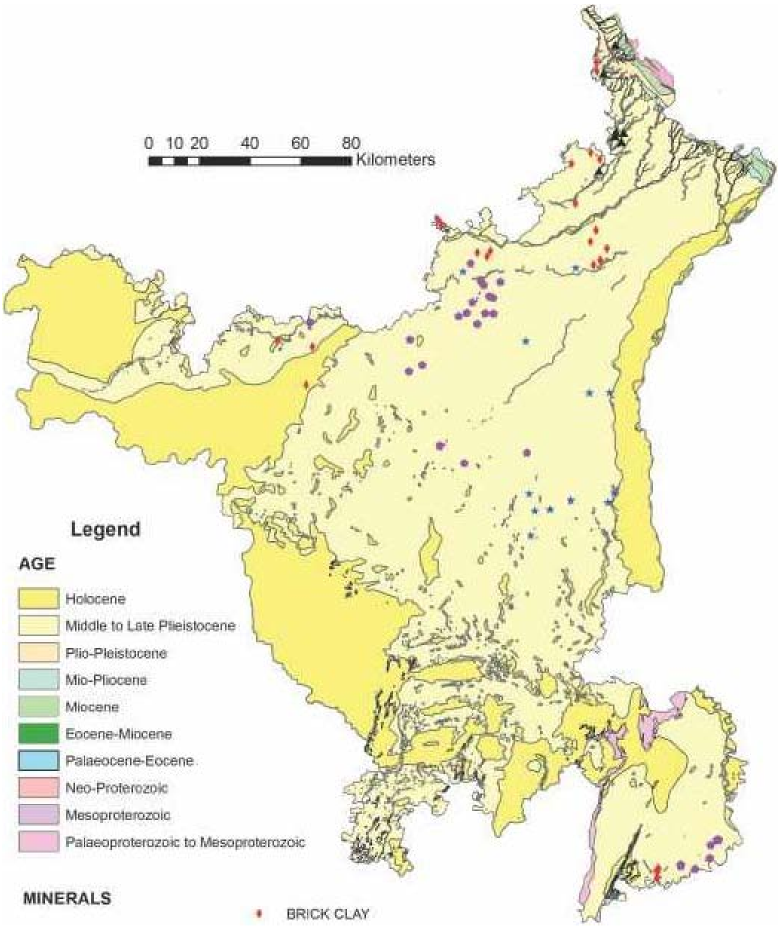

Fig. 4 Geomorphological Map of Haryana (GSI, 1973)

3. Depth of Ground Water Table

Liquefaction is most likely to occur in areas where ground water table lies within 10 m of the ground surface. There are few instances of liquefaction having occurred in areas with ground water table deeper than 20 m. Ground water conditions in State of Haryana have been analysed using ground water data collected by Central Ground Water Board, Chandigarh (CGWB, 2013) and have been mapped using nearest neighbour interpolation as shown in Figure 5.

In many districts of Haryana, the depth of ground water table is within the liquefiable zone i.e. less than 20 m. Also, some of the districts are dealing with the problem of subsurface water logging. These regions are highly susceptible to liquefaction during earthquakes. However, in district of Gurgaon, ground water table is 30 to 40 m below ground level, which makes the region very less susceptible to liquefaction.

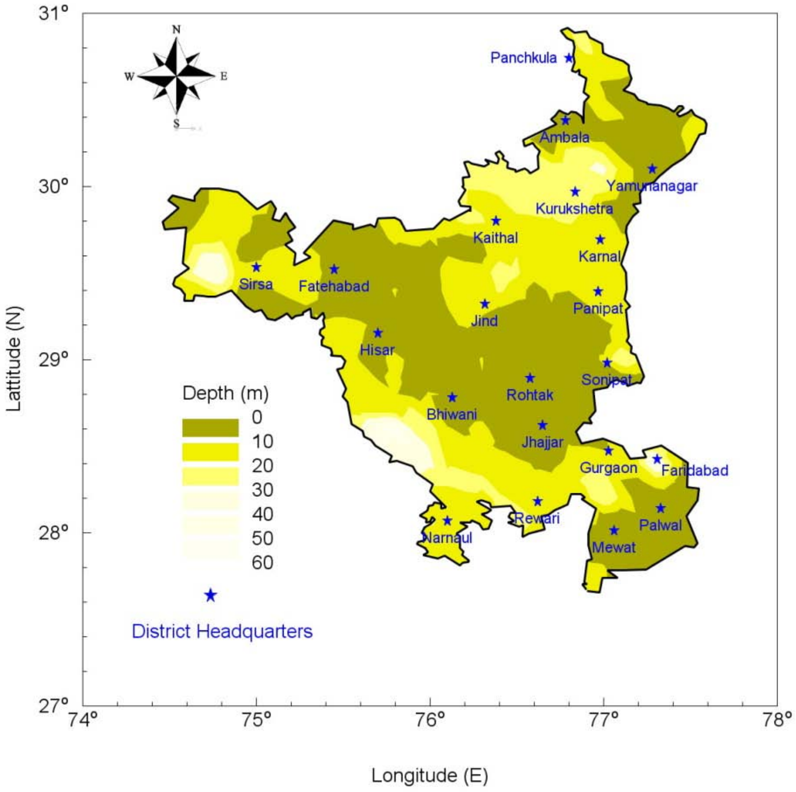

Fig. 5 Water Table Map

4. Seismic History of Haryana

In order to understand seismicity of Haryana, data regarding past earthquakes with magnitude ≥ 2.0 have been collected for a period of 55 years (1960-2014) from online portal of Indian Meteorological Department (IMD, 2014). It has been observed that during January1960 to November 2013, there have been 46 earthquakes in Haryana and nearby areas having magnitude (M) ranging from 2.3 to 6.0. Also, it has been observed that in districts of Sonipat, Rohtak and Jhajjar, maximum number of seismic events have occurred.

However, districts of Sirsa, Fatehabad, Hisar, Kaithal, Karnal, Panipat, Yamuna Nagar, Panchkula, Mewat and Palwal have been reported as seismically less active. In rest of the districts, few incidents of noticeable earthquakes have been reported.

No major earthquake has yet occurred in the state, but possibility is not ruled out as a large area of Haryana lies in Zone IV. Moreover, adjoining state of Delhi also falls in Zone IV and if any major earthquake occurs in Delhi, it would impact the surrounding area of 300 km. Experts predict that in the coming 50 years the region is bound to be hit by a severe earthquake of magnitude more than 6.0 on the Richter's scale. There is 80% probability of occurrence of an earthquake of the magnitude 7.0. This forecast is based on the detailed analysis of past earthquakes and underground movement of the region backed up by satellite imageries (Srow, 2013). Greater numbers of tectonic activities have occurred in Sonipat, Rohtak and Jhajjar districts during a short span of time, which makes them susceptible to liquefaction too.

5. Liquefaction Susceptibility of State of Haryana

Liquefaction susceptibility of the State of Haryana has been assessed by integrating the available information from earthquake hazard maps, flood hazard maps, ground water profile, lithological maps and other relevant reports of various government organizations. Various geomorphological, geological and seismic units have been identified in the study area. The analysis has been carried out following Saaty's

Analytical Hierarchy Process (Saaty, 1990). Analytical Hierarchy Process (AHP) is a multiple criteria mathematical evaluation method in decision making tools, specifically used for dealing with problems of spatial nature. A comparison matrix has been constructed on a scale of 1-3, 1 indicates that the two units are equally important, 2 shows that one unit is somewhat important than other and 3 implies that one element is moderately important than other. If an element is less significant than the others then it is indicated by reciprocals of 1-3 values (i.e. 1/1 to 1/3). The comparison matrix prepared for the study is reported in Table 6.

Table 6: Comparison Matrix

Units	Geomorphology	Lithology	PGA	Water Table
Geomorphology	1	1/3	1/3	1/3
Lithology	3	1	1/2	1/2
PGA	3	2	1	1/2
Water Table	3	2	2	1

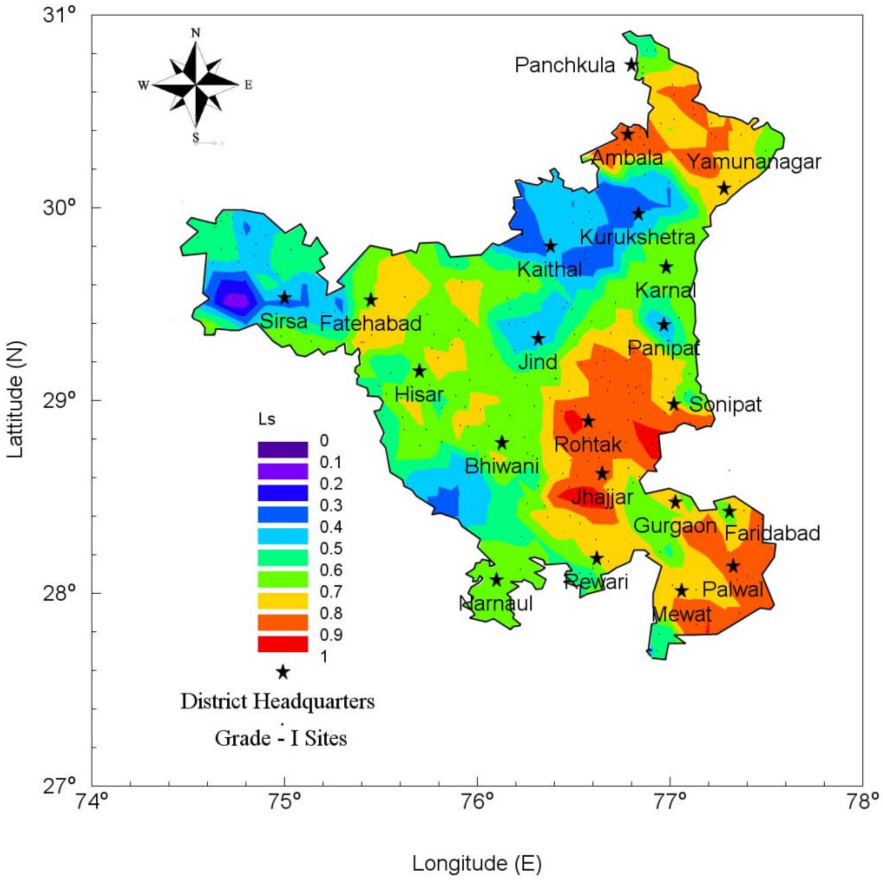

Fig. 6 Grade-I Liquefaction Susceptibility Map

Using the comparison matrix, weightage corresponding to each unit has been calculated. It is carried out by converting elements of comparison matrix into decimals and then calculating the principle Eigen vector of the matrix. This process is repeated until the Eigen vector solution becomes equal or very close to the previous iteration. The final Eigen vector represents the weights of different units. A value of 0.044 has been observed as consistency ratio, which shows that the weightages developed are very much consistent. The rating of features for each unit has also been normalized between 0 to 1 (Nath, 2004) to ensure that no unit exerts influence beyond its determined weightage. Influence factors corresponding to each feature (i.e. normalized ratings) have been calculated using the Equation (1):

$$x_i = \frac{R_j - R_{min}}{R_{max} - R_{min}} \tag{1}$$

Weightages of various units and influence factors of their features have been suggested in Table 7. Overall susceptibility to liquefaction has been determined in terms liquefaction susceptibility value ($L_s \leq 1$), which is simply summation of product of weightages of the units and influence factor of respective features Equation (2).

$$L_s = 0.2047\ I_L + 0.0965\ I_G + 0.4094\ I_W + 0.2895\ I_P \quad (2)$$

Where, I_L=Influence factor for lithology, I_G=Influence factor for geomorphology, I_W=Influence factor for water table and I_P=Influence factor for peak ground acceleration.

The state has been classified into three zones of liquefaction susceptibility viz., high ($L_s \geq 0.8$), moderate ($0.5 < L_s < 0.8$) and low ($L_s \leq 0.5$). However in case of clays and rocks susceptibility to liquefaction is always considered low irrespective of the value of L_s. The analysis has been carried out for 243 locations for which all required data were available and a Grade-I liquefaction susceptibility map has been prepared using nearest neighbour interpolation model as shown in Figure 6.

Table 7: Weightage, Rank, Influence factor and Susceptibility of Various Units and Features Identified in the Study Region

Unit	Weights	Features	Rating	Influence Factor	Susceptibility
Lithology	0.2047	Sands and Non Plastic Silts	3	1	High
		Loams	2	0.5	Medium
		Clays, Plastic Silts or Rock Outcrop	1	0	Low
Geomorphology	0.0965	Flood plains, river beds , Young deposits (age<500 years)	3	1	High
		Holocene deposits	2	0.5	Medium
		Pleistocene and older deposits	1	0	Low
Water Table	0.4094	0 to 10 m	3	1	High
		10 to 20 m	2	0.5	Medium
		>20 m	1	0	Low
Anticipated PGA as per IS:1893-Part 1	0.2895	0.24g, corresponding to zone IV	3	1	High
		0.16g, corresponding to zone III	2	0.5	Medium
		0.10g, corresponding to zone II	1	0	Low

It has been observed that districts of Ambala, Faridabad, Jhajjar, Palwal, Rohtak, Sonipat and Yamunanagar are highly susceptible to liquefaction during earthquakes. It is because these regions are basically flood plains with water table at shallow depth and fall in Zone IV of Seismic Zoning Map of India with maximum PGA of 0.24g. However, in these districts, areas with deep water table have been observed to be moderately susceptible to liquefaction.

The districts of Bhiwani, Fatehabad, Gurgaon, Hisar, Karnal, Panipat and Rewari have been observed to be moderately susceptible to liquefaction during earthquakes. This can be attributed to high depth of water table in these regions. Moderate risk of liquefaction in these districts can also attributed to the fact that these areas are not flood plains and their geomorphology varies from Holocene to Pleistocene and older deposits.

However, in above mentioned districts, regions with shallow water table and higher earthquake hazard have shown high susceptibility to liquefaction. In Panchkula district, gravelly soils are found in abundance and hence the region is moderately susceptible to liquefaction. Moreover, in Panipat City, susceptibility to liquefaction has been observed to be low. The districts Bhiwani and Fatehabad fall in Zone III of Seismic Zoning Map of India with maximum PGA of 0.16g. This is a contributing factor in moderate susceptibility to liquefaction of these areas.

Apart from this, in districts Jind, Kaithal, Kurukshetra, Mahendragarh, Mewat and Sirsa susceptibility to liquefaction is quite low. This can be attributed to high depth of water table in these areas. Moreover, a large area in these regions falls in Zone III of Seismic Zoning Map of India. Also, geomorphology for these areas varies from Pleistocene to older deposits.

VALIDATION OF RESULTS

Liquefaction susceptibility determined using proposed method has been randomly verified by analysing borehole data indicating Standard penetration test (SPT) values in the study area using semi-empirical procedure developed by Idriss and Boulanger (2006). The procedure is summarized below:

1. Appropriate soil type: Determine if the soil has the ability to liquefy during an earthquake.
2. Groundwater table: The soil must be below GWT. The liquefaction analysis could also be performed if it is anticipated that the groundwater table will rise in future, and thus the soil will eventually be below the groundwater table.
3. Cyclic stress ratio (CSR): Determine CSR that will be induced by the earthquake.
4. Cyclic resistance ratio (CRR): By using the standard penetration resistance test data, the CRR of the in-situ soil is determined.
5. Factor of safety (FOS): FOS=CRR/CSR. The higher the factor of safety, the more resistant the soil is to liquefaction. However, soil that has a factor of safety slightly greater than 1.0 may still liquefy during an earthquake. For example, if a lower layer liquefies, then the upward flow of water could induce liquefaction of the layer that has a factor of safety slightly greater than 1.0.

Typical boreholes showing location, depth and recorded SPT values have been shown in Figure 7. It has been observed that in most of the cases, areas identified using Grade-I technique as low, moderate and high susceptible to liquefaction, results given by semi-empirical procedure are quite comparable. The results have been reported in Table 8.

Fig. 7 Typical bore-logs showing soil classification and SPT values along the depth

Table 8: Comparison between Liquefaction Susceptibility Determined using Grade-I Technique and Semi- Empirical Procedure

Site No.	Location	Grade - I Technique		Idriss and Boulanger, 2006 (SPT Based Semi- Empirical Procedure)					
		L_s	Susceptibility	Depth for Minimum Factor of Safety (m)	Minimum C.S.R	Minimum C.R.R	Minimum F.O.S	Zonesof Liquefaction (m)	Susceptibility
1	Scout and Guide Hostel, Ambala	0.898	High	2	0.155	0.130	0.838	2, 9	High
2	R.O.B, Ambala City, Ambala	0.898	High	16.5	0.224	0.241	1.076	NIL	Moderate
3	Bus Stand, Tosham, Bhiwani	0.759	Moderate	1.5	0.103	0.131	1.273	NIL	Low
4	Sector 56, Faridabad	0.795	Moderate	10.5	0.139	0.150	1.079	NIL	Moderate
5	Labour court, Sector 12, Faridabad	0.795	Moderate	3	0.153	0.155	1.010	NIL	Moderate
6	R.O.B at BhattuMandi, Fatehabad	0.705	Moderate	6	0.131	0.167	1.272	18	Low
7	Sector 38, Gurgaon	0.392	Low	1.5	0.155	0.148	0.954	1.5	Moderate
8	NRFM&TI Training Centre, Hisar	0.657	Moderate	0.9	0.104	0.101	0.972	0.9	Moderate
9	Hansi, Railway Station, Hisar	0.657	Moderate	1.5	0.103	0.132	1.273	NIL	Low
10	PWD Rest House, Beri, Jhajjar	0.801	High	1.5	0.206	0.132	0.639	1.5 - 9	High
11	Sector 6, Bahadurgarh, Jhajjar	0.801	High	3	0.207	0.162	0.782	1.5 - 4.5	High
12	Railway Station, Jind City	0.452	Low	6	0.099	0.174	1.761	NIL	Low
13	Bridge on KalayatSajuma Road, Kaithal	0.452	Low	4.5	0.145	0.173	1.192	NIL	Low
14	Sector 13, Karnal	0.597	Moderate	4	0.174	0.137	0.788	4	Moderate
15	Village Kurali, Indri Road, Karnal	0.597	Moderate	0.9	0.156	0.157	1.007	NIL	Moderate
16	VibhutiMandir Complex, Kurukshetra	0.392	Low	24	0.117	0.119	1.020	NIL	Moderate
17	R.O.B on Markanda river, Shahabad, Kurukshetra	0.488	Low	3.5	0.152	0.133	0.877	3.5 - 15.5	High
18	STP, Narnaul, Mahindragarh	0.699	Moderate	3	0.102	0.172	1.69	NIL	Low
19	Medical College, Nooh, Mewat	0.699	Moderate	1.6	0.155	0.166	1.072	NIL	Moderate
20	Health Centre, Nagina, Mewat	0.795	Moderate	1	0.156	0.116	0.746	1 -2, 6-9	High
21	Government College, Sector 18, Panipat	0.392	Low	4.5	0.151	0.156	1.037	NIL	Low
22	IOCL Refinery, Panipat	0.801	High	10	0.248	0.162	0.653	7.5 - 10	High
23	Sector 17, Panchkula	0.591	Moderate	5	0.150	0.137	0.915	1, 3, 5	Moderate

24	Sector 21, Part 3, Panchkula	0.591	Moderate	4.5	0.151	0.480	3.183	NIL	Low
25	4G Telecom Tower, Hodal, Palwal	0.801	High	2	0.155	0.102	0.662	2, 16-20	High
26	Railway Station, Palwal	0.801	High	1.5	0.155	0.148	0.955	1.5	Moderate
27	I.T.I Tankri, Rewari	0.747	Moderate	1.5	0.155	0.187	1.204	NIL	Low
28	Pushpanjli Hospital, Rewari City	0.747	Moderate	12	0.169	0.143	0.842	1.5, 12 - 13.5, 16.5	High
29	R.O.B at Sampla, Rohtak	0.801	High	9	0.202	0.157	0.774	3, 6, 9 - 10.5	High
30	Sector 1, Rohtak City, Rohtak	0.801	High	9	0.276	0.153	0.555	1.5 - 10.5	High
31	Bridge Over Sheranwali Channel, Near Village Keshpura, Sirsa	0.355	Low	1	0.065	0.123	1.887	NIL	Low
32	R.O.B Near Railway Station, Sonipat	0.850	High	2	0.151	0.124	0.820	4.5, 24	High
33	4-Lane Bridge Near Village Barwasni, Sonipat	0.801	High	3	0.153	0.107	0.701	1.5 - 4.5	High
34	Sugar Mill, Bilaspur, Yamunanagar	0.801	High	12	0.219	0.162	0.742	6 - 12	High
35	SabziMandi, Yamunanagar	0.699	Moderate	4.5	0.151	0.141	0.933	4.5 - 6	Moderate

CONCLUSION

Grade-I liquefaction hazard mapping for the State of Haryana has been done on the basis of geological and geomorphological characteristics of the area. On the basis of that following conclusions have been drawn:

1. Soil resource maps, geomorphological maps, earthquake hazard maps and ground water table data have been used for assessing liquefaction susceptibility. The analysis has been carried out following Saaty's Analytical Hierarchy Process (Saaty, 1990). The state has been classified into three zones of liquefaction susceptibility viz., high $(L_s \geq 0.8)$, moderate $(0.5 < L_s < 0.8)$ and low $(L_s \leq 0.5)$. However in case of clays, susceptibility to liquefaction is always considered low irrespective of the value of L_s.
2. It has been observed that districts of Ambala, Faridabad, Jhajjar, Palwal, Rohtak, Sonipat and Yamunanagar are highly susceptible to liquefaction during earthquakes. It is because these regions are basically flood plains with water table at shallow depth and fall in Zone IV of Seismic Zoning Map of India with maximum PGA of 0.24g.
3. The districts of Bhiwani, Fatehabad, Gurgaon, Hisar, Karnal, Panipat and Rewari have been observed to be moderately susceptible to liquefaction during earthquakes. This can be attributed to high depth of water table in these regions. In Panchkula district, gravelly soils are found in abundance and hence the region is moderately susceptible to liquefaction.
4. In districts Jind, Kaithal, Kurukshetra, Mahendragarh, Mewat and Sirsa susceptibility to liquefaction is quite low. This can be attributed to high depth of water table in these areas. Moreover, a large area in these regions falls in Zone III of Seismic Zoning Map of India. Also, geomorphology for these areas varies from Pleistocene to older deposits.
5. Liquefaction susceptibility determined using proposed method has been verified by analysing boreholes in the study area using semi-empirical procedure developed by Idriss and Boulanger (2006). It has been observed that in most of the cases, areas identified using Grade-I technique as low,

moderate and high susceptible to liquefaction, results given by semi-empirical procedure are quite comparable.

6. Regional studies based on geological and geomorphological data would not be a substitute to detailed site specific investigation, but could indicate areas where thorough investigation is required.

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