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VOLUME VIII
THEME 6
Seismic and seismotectonic investigations of engineering projects

THEME 7
History and development of Engineering Geology

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VOLUME VIII

THEME 6
Seismic and seismo-tectonic investigations of engineering projects
Investigations seismique et seismo-tectoniques des projets de l’ingénierie

THEME 7
History and development of engineering geology
Histoire et développement de la géologie de l’ingénieur
The Organising Committee of the 4th International Congress of Engineering Geology feels particularly happy to be able to release in time the nine volumes of the Proceedings of the Congress, running to about 3000 pages. The first eight volumes encompass over 270 scientific and technical papers, one-third of which are from India and two-thirds from 32 other countries of the world. The ninth volume contains the Panelists' Reports, the Sessional Reports and the Special lectures planned to be delivered during the Congress. The tenth volume, containing the record of the Technical Sessions and the discussions to be held during the Congress, is proposed to be released at a later date.

The seven Themes as chosen for discussion during the 4th Congress cover a wide range of topics, all of which are of special interest to the present-day developmental activities in the world. The listed Themes are:

2. Engineering Geological Problems of Tunneling and Excavation of Cavities;
3. Soil and Rock as Construction Material;
4. Engineering Geological Problems of Natural and Man-made Lakes;
5. Engineering Geological Problems of Sea-coast and Shelf Areas;
6. Seismic and Seismo-tectonic Investigations of Engineering projects; and

The themes as listed above have been further sub-classified into 31 sub-themes, so as to bring into their fold several related problems and to focus attention on specific aspects.

Although the Organising Committee feels very much gratified by the overwhelming response to the call for scientific and technical papers for discussion during the 4th Congress, and nearly 300 papers were received from different countries of the world, the major and somewhat perplexing task faced by the Committee was to classify the papers received and to pigeon-hole them into the various Themes and sub-themes. In fulfilling this onerous task, the Committee has done its best in good faith and has offered a classification in the eight volumes of the Proceedings which may be taken as one of the best-fits for the Themes and the sub-themes and not as the only best possible fit. Again, by stretching the ambit of some of the themes, a number of papers, which were of great topical interest and were considered to be of an auxiliary nature as well as related to the Themes posed for discussion, were accommodated, as is particularly the case with Theme 1. In some of the papers, minimal language corrections were made, taking care, however, to retain the style of the authors' presentations to the maximum possible extent.

A very large number of papers were received for Theme 1 and these have been distributed in the first three volumes: the first volume containing all the papers falling "sensu stricto" under Theme 1 while Volumes II and III containing all the papers of an auxiliary or supporting nature and which are, in one way or the other, related to Theme 1. Theme 2 had also more than 50 contributions; therefore, these have been accommodated in two volumes, Volumes IV and V. While the papers for Theme 3 have been included in Volume VI, the papers for Themes 4 and 5, put together, have been incorporated in Volume VII and those of Themes 6 and 7, put together, in Volume VIII. The Organising Committee thanks all the contributors for their generous cooperation. The Committee is grateful to the various National Groups for the trouble they had taken to collect the papers from their respective countries and to forward these to the Organising Committee for its consideration. The Geological Survey of India has been the backbone of all activities relating to the Congress and the work of publication of the Proceedings of the Congress has not been an exception in its contribution. The Organising Committee is under a deep debt of gratitude to the Director General, Geological Survey of India and to the Officers working under him in different geoscientific disciplines, for their generous and unflailing support.

The main burden of the review of all the papers and the editing of some of them was shared by M/s.

New Delhi
12 September 1982

The Organising Committee is obliged to all these geoscientists for their unremitting help and support.

M/s. Oxford & IBH Publishing Co., New Delhi, did a commendable job in bringing out all the volumes of the Proceedings of the Congress in very good shape and in good time, despite several odds faced by them. They deserve the sincere thanks of the Organising Committee.

V.S. Krishnaswamy
Chairman, Organising Committee
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SEISMIC AND SEISMO-TECTONIC INVESTIGATIONS OF ENGINEERING PROJECTS
ENGINEERING GEOLOGY IN SEISMIC MICROZONING

LA GEOLIGE DE L'INGENIEUR DANS LA MICROZONATION SISMIQUE

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ABSTRACT

Microzoning, as is well known, is a technical procedure aimed at qualifying and, as far as possible, quantifying the elements for the planning of engineering works in seismic areas. For the statistical study of recorded earthquakes, geological and seismotectonic investigations, the analysis of the geophysical and geotechnical properties, systematic survey of construction damage and the evaluation of local seismic response, the collaboration of specialists from different fields is required in order to establish the suitable technical standards for the areas of interest. In this approach, the contribution of engineering geology is essential, being the discipline that acts as a hinge between the different specializations for the finalization of the general geological information, necessarily descriptive, utilized in the definition of the geotechnical parameters for engineering designing in seismic areas. In the present work, also on the basis of the authors' recent experiences, the different aspects of specialist intervention are examined and the operational criteria are discussed. The sooner these are adopted, the more evident the benefits in terms of minor damage to persons and things.

ABSTRACT

La micronization est notoirement la procédure technique qui vise à donner les éléments qualitatifs et, autant que possible, quantitatifs pour les plans des travaux de l'ingénieur dans les régions sismiques. L'étude statistique des tremblements de terre enregistrés, les investigations géologiques et sismotectoniques, l'analyse des propriétés géophysiques et géotechniques, le relief systématique des dommages aux constructions et la evaluation de la réponse sismique locale, requiert la collaboration de spécialistes de différents domaines au fin de prédisposer des règles techniques appropriés aux régions de l'intérêt. Dans cette approche la contribution de la géologie de l'ingénieur est essentielle, étant donné que c'est la discipline qui agit comme une charnière entre différentes spécializations pour finaliser les informations de la géologie générale, nécessairement descriptive, à la définition des paramètres géotechniques pour les projets de l'ingénieur en régions sismiques. Dans ce travail on examine les différents aspects des interventions spécialistes même sur la base de l'expérience personnelle des auteurs, et on discute des critères opératifs. Autant plus opportuns seront les critères, autant plus les bénéfices seront évidents en termes de dommages à personnes et à choses.
1. INTRODUCTION

According to recent experiences in Italy and other countries, the technical problems encountered in seismic areas are so complex that interdisciplinary groups of experts and technicians are always required in order to give the administrators quantified alternatives and the data necessary for the planning of interventions.

As is well known, the intervention in a given area should be preceded by at least:
1. A first zoning level, where the characteristics of probable earthquakes are defined;
2. A second zoning level, where the influence of local conditions is evaluated;

Zoning problems have been debated in several international meetings in the last decades and especially in the 1st and 2nd International Conferences on Microzonation held in Seattle (1972) and S. Francisco (1978) and recommendations have been proposed at the Intergovernmental Conference on the Assessment and Mitigation of Earthquake Risk, convened by UNESCO (1978).

Fruitful experiences have also been derived from the latest serious earthquakes in Italy, when the technical and scientific services of the country were mobilized in order to find a procedure suitable for the local situations (BRAMBATI et al. 1980, I.G.L., 1981).

As far as the first zoning level is concerned, most experts concur on the type of basic information, on the procedures to be adopted and on the representation of data. On the other hand, several, even quite different, methods have been proposed for the more detailed zonation or microzoning and a unified approach does not as yet exist.

A discussion on the principal methods adopted in various countries and especially in Japan, USSR and USA is outside the interests of the present paper which is mainly devoted to underlining the specific role and that of coordination which may be played in microzoning by engineering geology, with respect to geology, geophysics, geotechniques and structural engineering. In fact, in these specific fields the absence of suitable coordination on the one hand and the excessive specialization and lack of expertise of some specialists in sectors other than their own, on the other, might provide planners and administrators with both insufficient and redundant information, especially when, for various reasons, data have only been collected in emergency situations and studies and investigations have not been sufficiently well-organized.

2. ZONING LEVELS

Obviously, for a correct and effective zoning, the knowledge of geostuctural and geomorphological characteristics that determine the physical medium affected by the earthquake, is essential. However, the general geological studies must be followed up by interdisciplinary technical and socio-economic investigations, in order to quantify the design parameters for correct planning and regulations at the different levels.

The interaction of the different phases preliminary to the operations is schematically represented in the flow-chart of Table 1, in agreement with the present orientation on the specific theme of microzoning considering the recommendations of UNESCO (1978) and the "Guide to Microzoning investigations" prepared, but not yet published, by the Working Group on Seismic Microzoning of the Finalized Project "Geodynamics" of the National Research Council (FACCIOLI, 1980).

Obviously, not all the operations shown in Table 1 are always essential. They may be developed in a different way and some may even be omitted in function of:
1. The extent of the area involved;
2. The availability of instrumental data and
3. The investigations already carried out.

The results of the first level zoning are represented in seismicity maps generally at a small scale. In densely populated areas such as Italy, where, with old villages often situated on unstable slopes, earthquakes even of moderate intensity are likely to be destructive, the preliminary classification of sites should also take into
TABLE 1. Interrelation between first and second level zoning and planning.
account the propagation path models realized on the basis of near field and source mechanism effects.

It is the general opinion that microzoning studies should involve areas no larger than a city and aim at providing the engineering design parameters, represented with some estimated probabilities of exceedance in maps of a suitable scale, in any case no smaller than 1:25,000 and preferably larger. In reality, it might be necessary to extend the investigations beyond the boundaries of the city areas, which do not often coincide with relevant details such as fault zones, geological bodies and overburden and bedrock morphology.

In any case microzoning studies should be planned on the basis of a good knowledge of the seismotectonic characteristics of the region under consideration, represented in maps at a scale not exceeding 1:100,000. In fact the different zoning phases are so closely associated that if the knowledge of the general geology and the geotechnical properties is insufficient, all the subsequent steps may be rendered invalid and the effectiveness of the interventions may be seriously affected. The sequence is iterative and the data of each step is the result of adjustments determined by the contribution of different specialists.

In this perspective, since engineering geology covers an area that partly overlaps those of the geological and mechanical disciplines, it represents the natural link between geologists, geophysicists, geotechnicians and structural engineers for the planning and interpretation of the results of the investigations aimed at identifying the geometrical and dynamic site and rock characteristics.

3. ENGINEERING GEOLICAL INVESTIGATIONS

Owing to the interdisciplinary nature of the field it covers, engineering geology may certainly represent an advantageous contribution at all levels of seismic zoning. Nevertheless, its role seems essential in the transition between the first and second level, when, depending on available information and funds, the investigations should be planned and executed for the punctual definition of local seismic effects and all data should be combined for the purpose of identifying the critical areas.

In order to avoid dissipation of human and economic resources, it is indispensable that the geological, geophysical and geotechnical measurement campaigns are well organized and the investigations restricted to the achievement of the intermediate and final targets, which are substantially:

1. The tridimensional geometrical definition of homogeneous blocks;
2. The identification of the dynamical properties of soils and rocks that constitute the blocks and
3. The classification of zones with similar exposure to earthquake effects.

The problem is particularly difficult in geologically complex areas as recently seen in Friuli and in Southern Italy. There the earthquakes of 1976 and 1980 had marked effects on the slope stability and aquifer regimes, with consequences also on water supplies.

The achievement of the three targets implies:

a. The collection of existing data (seismic records, geological, hydrogeological and stratigraphical surveys, geotechnical properties measurements in situ and in the laboratory, etc.);

b. The planning of direct investigations;

c. The elaboration of all the data collected.

Documentation already available provides useful assistance in the planning of geological and geotechnical investigations, especially in urban areas where only rarely geology can be studied in situ. Only after careful analysis of all the data available, which in older cities may even include historical documents, can drilling and excavations or geophysical measurements be planned. Nevertheless, in order that existing information can be utilized profitably, standard collection criteria and specific regulations should be incorporated in building codes, as adopted in Mexico by the Secretaria de Obras Publicas (1976). Moreover, data are generally stored in computers, but for safety reasons, especially
TABLE 2. Engineering geology investigations.
in urban areas it may be advisable to preserve a copy of the original documents in a different site, in order to ensure their availability even in the case of damage to the data bank.

The field and laboratory studies are carried out in particular at the second level of zoning, but in seismic areas geological and geotechnical data from every available source should be collected already at the first level and some simple tests planned at least for the measurement of the index soil and rock properties (grain size, Atterberg limits, uniaxial compression, etc.).

In Table 2, which develops the sector with the dotted line of Table 1, the main elements that should be identified in order to estimate the local seismic ground response are summarized. Obviously the data should be fitted to the geological characteristics and to the socio-economical importance of the area. In any case, not only from a geological point of view, but also as far as the general reliability of the results of microzoning is concerned, an understanding of the geometrical, geostuctural and geomechanical characteristics of the bedrock as well as the location, thickness and stratigraphy of overburden materials, is of paramount importance (CLUFF, 1978). The analysis of the local seismic response by means of the usual calculation programs, such as the finite element method, requires in fact that the dynamic characteristics of the deposit are defined in the various points of the grid under consideration and moreover that all boundary conditions are known. The more certain the geometric and geomechanic parameters of the site, the more reliable the results of the model. Thence, also the calculation models of soil-structure interaction would be better defined and more correct.

The scheme of Table 2 depicts the close link between the different disciplines and their complementarity in planning and interpreting the results of investigations. The prominent role of the engineering geologist in coordinating and analyzing all the results of the studies aimed at identifying critical areas, is also evident. Geotechnical and geophysical measurements must be based on a good knowledge of the local geology and particularly of the existing and potential geological hazards either natural or induced by man. Moreover, simulation by means of laboratory geotechnical tests requires the identification of the main factors that affect the dynamic response of the site. The influence of elements such as the anisotropy of deposition in situ, the entity of the K0 coefficient, the effects of cyclic loading and unloading, the shape and packing on saturated sands, the influence of multidirectional shear, etc. can only be studied in the light of the local geological situation.

The investigations should be concentrated in critical areas and repeated in the blocks identified as homogeneous. For a statistical treatment of data within the different blocks, the number of samples should be proportional to the extent and variability of the properties. Statistical techniques of quantitative classification of sites, such as factor and cluster analysis, can provide advantageous contributions to the subdivision of a region into zones with similar exposure to earthquakes (CRESPELLAN, 1979, CRESPELLANI, LOI, 1979). Therefore the number of investigations and of samples should be determined beforehand.

All the results of laboratory and field investigations and of geostatistical analysis, could be represented in detailed engineering maps, at the scales of 1:5,000 or 1:10,000, where the lithotypes in areas of major risk, such as old and new landslides, existing or potential, and areas susceptible to liquefaction, are identified according to the mean values and standard deviation of the principal geotechnical properties. The maps should be accompanied by cross sections with the mean values and the law of variation of geotechnical properties with depth. The main geotechnical parameters should also be correlated on a regional scale. In these maps all the data necessary for the evaluation of seismic hazards must be indicated and all elements for the successive steps defined.

The engineering geological investigations should be carried out in connection
with the structural studies on old and new buildings and their foundation systems. Particularly important is the correlation of damage maps from recent earthquakes and geological and geotechnical maps in order to identify the causes of the damage and to forecast similar situations. For this comparison more detailed geotechnical maps (also in scale 1:2,000 or 1:1,000) could be necessary and unified criteria of mapping should be adopted. On the occasion of the last earthquake in Southern Italy in 1980 a remarkable effort was made in this direction but evidently the planning of investigations and the proposal of criteria should be arranged before critical events.

4. ZONING AND PLANNING

The output of the second level zonation is the evaluation of the seismic risk, defined on the basis of the elements by:
1. The mapping of the damage;
2. The topographical, geological and geotechnical characteristics and;
3. The definition of the local seismic effects, considering also the socio-economical situation.

The seismic risk maps represent the basis for urban planning and consequently for the decisions concerning regulations and interventions. Of course, these maps should be neither definitive nor require long elaboration. Preferably, they should be computerized so that they can easily be kept up to date with Bayesian type techniques.

It is the administrators' responsibility to predict the interdisciplinary groups of technicians when planning the civil defence and not in emergency situations. All the investigations relating to the zoning take time as does the preparation of experts. Specialists such as engineering geologists, geologists, geophysicists, geotechnical and structural engineers, urbanists, sociologists and economists whose collaboration is essential, cannot be easily found on demand everywhere. In microzonation urban areas the collaboration of experts in statistics and informatics is also necessary.

In an interdisciplinary group the number of experts could vary in function of:
- the extent of the area of investigation and the type of settlement;
- the social and economical situation;
- the funds at disposal for the research;
- the state of scientific knowledge of the area;
- the time.

In any case, an integrated study by various specialists appears to provide a correct approach to microzoning, on condition that the different aspects of the seismic hazard are coordinated in a general view.

5. CONCLUSIONS

Zoning at different levels is always a complex operation requiring many specialists with different competences. Researchers are concentrating on solving the problems, mainly involved with collecting information suitable for elaboration. In this context the contribution of engineering geology may be decisive, not only as regards investigations in this specific field, but also its role of coordination between the different disciplines.

SELECTED REFERENCES


RESULTS OF CRUSTAL MOVEMENT STUDIES IN INDIA

LES RESULTATS DES RECHERCHES DES MOUVEMENTS DE L’ECORCE EN INDE

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ABSTRACT

India is a country with very complex and divergent geographical and geological features with numerous major and minor faults/thrusts, tectonic foldings, plateaus, plains, and hill ranges of varying heights. The study of crustal movements in India can be considered to date back to 1920s when subsidence of various bench-marks was studied with the help of spirit levelling. Recently the scope of such studies has been extended to cover also variations in plan positions and allied geophysical studies. The importance of crustal movement study has been brought to focus by several Hydro-electric projects being planned in the country. Invariably these project areas are located astride faults/thrusts and it becomes essential to study crustal movements and to see what corrective steps could be taken in the project design to ensure the safety of large engineering structures. There are several other areas of scientific investigations such as earthquake prediction in which the study of the crustal movements form a very helpful complement.

In this paper an attempt has been made to indicate the regions of our country in which such studies have been taken up. Results of the completed studies have been given and future plans have been outlined.

ABSTRACT

L’Inde est un pays avec des traits géographiques et géologique - tous très compliqués et divergents; avec beaucoup de failles, les pousiers, les deux majeurs et de second ordre, et les piliers tectoniques, les plateaux, les plaines, et les chaînes des montagnes d’élévations changeant. En Inde on peut considérer que la recherche de mouvements de l’écorce commença à 1920 quand l’affaissement de repères différents était étudié avec l’aide d’un niveau à bulle d’air. Recemment les carriers de ces études a été prolongé pour couvrir aussi les variations aux positions de projet et les recherches aux géophysiques alliées L’importance de la recherche de mouvements de l’écorce a mis en question a cause de plusieurs projets concernant Hydro-électrique, pour lesquels le gouvernement prépare le système ice en Inde. Invariablement, on peut trouver ces régions de projet qui couvrent a construire, a califourchon des failles/les poussées, et il faut étudier les mouvements de l’écorce pour découvrir quelles mesures correctives, on peut considérer concernant le dessein pour le project pour garantir la sécurité de grandes constructions de l'ingénieur. Quasi, il-y a quelques autres régions aux fins d'enquête scientifique comme la prediction d'un tremblement de terre dans laquelle la recherche de mouvements de l'écorce peut fournir un complement très utile.

Dans cet expose on a fait un attretat pour indiquer les régions de notre pays au on a pris ces recherches. Nous avons compris taut les résultats des recherches achevées avec les contours de nes projets à l’avenir.

VIII.11
1. INTRODUCTION

The phenomenon of earth’s crustal movements caused by processes such as faulting, bending, folding and over thrusting and the contribution of geodesy in monitoring these are well known. The main problem of the geodesist has been to detect crustal movements from the data of repeat geodetic observations, after allowing for observational errors and local subsidence such as alluvium shrinkage, soil creep and land slides.

India is a country with a very complex and divergent geographical and geological features with numerous major and minor faults/thrusts, such as Himalayan tectonic foldings, High Ladakh Plateau, Peninsular Shield, Shillong Masif, Saurashtra-Kutch Shelf, Gondwana and Guddapah basins, Gangetic plains, Narmada-Son Lineament, Vindhya and Aravalli ranges, and Steep Western Ghats. There are also gigantic dams which are suspected to cause variations in strains due to the reservoir loads.

Geodetic and Research Branch of the Survey of India, the national mapping organisation, carries out Crustal Movement Studies by geodetic and geophysical methods in some selected areas in connection with Engineering and Hydro-electric Project surveys and some areas of special interest. Other organisations which are participating in such studies are:

i) Geophysical Division of the Geological Survey of India (by geophysical methods)
ii) National Geophysical Research Institute (by geophysical methods)
iii) Roorkee University (by geophysical methods)
iv) Indian Meteorological Department (by seismological observations)

The ensuing paragraphs give an overall view of all the geodetic and allied geophysical investigations carried out in India for evidence of recent crustal movements.

For this purpose India has been considered in the following regions for convenience:

i) Northern Zone including Northern & Western Himalayas

ii) Central Himalayas and the Indo-Gangetic Plain

iii) North Eastern Zone including the Eastern Himalayas

iv) Central Zone including the Aravalis and the Narmada-Zone Rift Zone

v) Southern Zone

2. NORTHERN ZONE (NORTHERN & WESTERN HIMALAYAS)

2.1 Levelling Line Dehra Dun - Saharanpur

The line from Dehra Dun - Saharanpur had been levelled in 1861-62, 1905-07, and 1927-28. A rise in the height of benchmarks up to the extent of 16 centimetres was noticed. The rise is not very phenomenal. It was, however, presumed that the rise in 1905-07 was due to Kangra Valley earthquake of 1905 and the further rise in 1927-28 is due to rise of Siwaliks and of Dehra Dun over Saharanpur regions.

2.2 Levelling Line Saharanpur - Ambala

The Garrison Engineer Ambala had reported that there was subsidence of ground near the Standard Bench-Mark situated in the compound of St. Paul's Church, Ambala Cantonment. A releveling of the line indicated that there was apparent rise of level by 2 to 5 centimetres up to Ambala and then a slow fall by similar amount. This was not a settlement of a few isolated bench-marks but a general sinking (although unequal) of many benchmarks through several miles of lines. It was concluded that the country around Ambala had sunk 2 to 5 centimetres since 1910-15, in continuation of the sinking of 18 cm from 1860-62. The apparent rise between Saharanpur and Ambala is more likely to be due to subsidence of both these places than due to actual rise of the intermediate country although the latter is a possibility. The Director of Geological Survey had expressed the opinion that the subsidence is more likely to be due to waterlogging of wells than to general earth movement.

2.3 Jammu Kashmir and Himachal Pradesh

7 geotraverse profiles were run for the Geodynamics studies in Kashmir and Himachal Pradesh. Various gravity geomagnetic and Hayford deflection anomalies were computed, geoid and compensated geoids and other anomaly etc. Charts with profile were prepared and interpreted. High positive Isostatic anomalies over the great Himalayan Range and negative anomalies in the Karakorsams showed that complete isostatic equilibrium did not prevail in the region. The geomagnetic anomalies
indicated a zone of recent tectonic formation. Both gravity and geomagnetic anomalies showed steep gradient in the vicinities of the Main Boundary Fault, Main Central Thrust and the Indus Suture line and also near other numerous thrusts and faults. An invisible channel of low density was found to have existed passing through the Punjab, Haryana and Uttar Pradesh below the foot hills of Himalayas and south of which in the opposite direction. The isostatic compensated geoidal undulations indicated the same pattern. The values of mean Hayford isostatic deflection anomalies lend support to the view that isostasy in long wave length was generally in existence and that the Himalayas were generally very much under compensated.

2.4 Garhwal and Kumaun Himalayas in Uttar Pradesh

3 geotraverse profiles were run in the area across some known and delineated faults/thrusts. The trend in the various gravity anomalies near and parallel to the faults/thrusts confirmed trend found at other places. Irregularities in the anomalies were also found at other places suggesting the existence of activity astride some unknown and unmarked faults/thrusts.

2.5 Himalayan Peaks

The heights of the Himalayan Peaks - Bandarpunch, Srikanta, Jaonli and Kedarnath were determined by vertical angle measurements in 1905 applying refraction correction. The determination was repeated in 1932. At that time no appreciable and systematic change was found. The observations were repeated during 1974-76. The geoid was extended with fresh and denser observations. Closer vertical angles were observed, similar to the procedure adopted for determination of height of Mt. Everest. The heights were revised with variation in height by +5.0 m, +3.0 m, -6.0 m and +27.0 m respectively. Kedarnath height having the variation of +27.0 m, the point of observation was also changed in this case. These variations, however, do not indicate the secular change in heights but could be due to other factors e.g. modern techniques of observations and reductions, observation from a close point and the change of the geoid with additional data.

Similar observations were also carried out for the peaks of Trisul and Nanda Devi. The height of Trisul was revised by +18.0 m but in case of Nanda Devi, the old height was retained.

The peaks of Makalu, Jano, Kanchan Junga, Tolung and Kabru were also reobserved during 1976-77 from Darjiling employing similar techniques as above but no appreciable and systematic change was noticed to warrant the change in their pre-determined heights.

2.6 Ganga Tear Fault in Uttar Pradesh

Geodetic triangulation, levelling and base measurement were carried out for the crustal movement studies near Haridwar across Ganga Tear Fault during 1974-76. The scheme was revised during 1978-79 except levelling observations. The horizontal movement of 1.0 cm to 5.6 cm was revealed during 1974-76 to 1978-79.

2.7 Shali Thrust in the area of Shanan (Joginder Nagar) in Himachal Pradesh

Repeat observations involving geodetic triangulation and precision levelling were carried out by No. 14 Party of the Geodetic and Research Branch across a suggested position of Shali Thrust near Shanan during 1975-76 and 1977-78 for recent crustal movements.

From an analysis of the results obtained for these two seasons, it is inferred that horizontal movements of the order of 0.5 cm to 4 cm and vertical movements of the order of 1 cm to 5 cm have taken place during the two years interval between the observations.

Based on the above observations and other data (still under analysis) a study is in hand to evolve a composite picture of geodynamics in the Western Himalayas.

2.8 Krol and Nahan Thrusts in Yamuna Hydel Project area in Himachal Pradesh

Angular measurements were taken in 1969-70 and repeat measurements during 1970-71, 1971-72, 1973/74 and 1975-76. Dense gravimetric observations were also carried out during 1974-75. The rate of movement was found to be 4 mm per year from SE to SW, the dips of thrusts 25° to the North and depths of these formation 2 km.

2.9 Across Yamuna Tear Fault in Yamuna Hydel Project Himachal Pradesh

The geodetic angular, base and levelling observations were first carried out during 1960-61 near Pacts. The levelling observations were repeated in 1972-74 and 1975-76 and others during 1965-66, 1972-74 and 1975-76. No significant velocity of vertical movement was revealed by a statistical analysis of results. The horizontal movement in some
of the Pillars was from 7 mm to 57 mm.

2.10 Main Himachal Thrust in Uttar Pradesh

Two quadrilaterals were established by geodetic triangulation to study the crustal movements across the Main Himalayan Thrust near Uttarkashi in 1973 in connection with proposed Pala-Manari Hydro-electric Scheme. Gravity observations were also taken on the stations. These were also connected by levelling. Triangulation and levelling observations were repeated during 1976-78. Horizontal movements in some of pillars was from 3 cm to 15 cm and vertical movement was found 0.7 cm to 8.8 cm.

2.11 Srinagar Thrust in Uttar Pradesh

Two quadrilaterals were established by Geodetic triangulation to study the crustal movements across Srinagar Thrust in 1973-74 alongwith gravity and levelling connection in connection with Manari-Bhali Hydro-electric Project Stage-II. The geodetic triangulation and levelling observations were repeated during 1976-78. Horizontal movements in some of pillars was from 4 cm to 5 cm and vertical movement was found 2 cm to 6 cm.

2.12 Across Suspected Beas River Fault and Satelitta Thrust

Geodetic triangulation observations were carried out during 1960-61, 1965-66 and repeated during 1974-76 covering area of 45 sq km for crustal movement studies. The analysis of the results indicated a movement in two pillars by 4.3 cm and 6.6 cm during the period 1960-61 and 1974-76. An additional scheme was included during 1974-75 by establishing 6 new crustal movement pillars across Satelitta Thrust and 8 Crustal Movement Pillars across Beas River Fault for repeat observations subsequently and for better determination of movements.

The geodetic stations and some other bench-marks were connected by levelling during 1960-61 and 1973-74. No significant velocity of movement was revealed in the statistical analysis of the results.

2.13 Levelling line Pathankot - Dalhousie (86 km)

The levelling was carried out during 1960-61 and repeated in 1972-73. One bench mark on the line has shown a velocity of vertical movement of 12.1 mm/ year and the rate was found to be increasing towards Dalhousie.

2.14 Levelling line Pathankot - Dharamshala (90 km)

The levelling was carried out at the two epochs of 1909-10 and 1918-19. The maximum velocity of vertical movement of 7.8 mm/year was found at one of the benchmarks in the negative direction.

2.15 Levelling line Jammu - Srinagar (294 km)

The levelling was carried out during 1922 and repeated during 1975-76. The maximum velocity of vertical movement of 1.8 mm/year was found at one of the benchmarks.

2.16 Levelling line Ghaggar - Simla (91 km)

The levelling was carried out during 1910-14 and repeated during 1975-76. No significant velocity of movement was revealed in the statistical analysis of the results.

2.17 Levelling line Dehra Dun - Harbartpur via Rajpur and Mussoorie (105 km)

The levelling was completed in 1975-76 in continuation of levelling portion Dehra Dun - Mussoorie completed in 1974 which was levelled during 1903-05, 1905-06, 1926-30 and 1952. The repeat levelling has shown a progressive rise in benchmarks of 6.8 cm between 1905 and 1974 up to a distance of 14 km from Dehra Dun and 13.8 cm rise in a bench-mark at Dehra Dun. The rise of bench-marks decreased from 6.8 cm to 1.0 cm up to a distance of 25 km between 1929-30 and 1974 and thereafter again rose to 9.8 cm between 1905 and 1974.

2.18 Levelling line Kotdwar - Landstone (42 km)

The levelling was revised during 1975-76. Earlier this line was levelled during 1909-10. The investigations and the statistical analysis of the results indicated that there was no appreciable motion of the bench-marks in the vertical direction.

2.19 Levelling line Ranibagh - Nainital (50 km)

The levelling was carried out during 1975-76. Earlier a portion of this line of 14 km was levelled during 1909-10. The analysis of results have shown velocity of vertical movement of 4 mm per year in the downward direction on one of the benchmarks.
3. CENTRAL HIMALAYAS AND INDO-GANGETIC PLAINS

3.1 Levelling line Dinajpur to Purnea

The line Dinajpur–Purnea and Bagaha–Siranj were relevelled in 1934–1935 in order to determine the disturbance, if any, of the bench-marks at Purnea and Bagaha which were terminals of the levelling done in North Bihar in connection with the earthquake of 1934. The height of Bench Marks at Purnea as determined from Dinajpur agreed to within 8 cm with the pre-earthquake height. The height of Bagaha showed a rise of 38 cm to 46 cm as determined just after the earthquake in 1934. This showed that Dinajpur itself had sunk by about 46 cm.

3.2 Eastern India

Ranchi Plateau, Chhotanagpur and Singhbhum area falls under this region. Levelling line and Geo-traverse profiles have been established in the region during 1976–78 for repeat observations subsequently for crustal movement studies.

A portion of the levelling line established during 1920–21 and repeated in 1965–66 was studied and result analysed. The velocity of vertical movements of different bench-marks was different, some indicating sinking and others rising trends.

The Chilka lake area was also studied for the structure indicating that they represent stable assemblage of the Sillimanite zone.

The levelling of 1948–49 and 1956–56 to connect the Sagar Island with the mainland levelling near Calcutta has indicated the subsidence of Sagar Island by about 0.2 m in a period of 7 years. Since this involved River Crossings across Wide Creeks, the discrepancy may also be due to error of observations. Repeat levelling would give precise determination of the crustal movements in the area.

4. NORTH EASTERN ZONE AND EAST HIMALAYAS

4.1 North Eastern Himalayan Region

Geodetic and geophysical observations are in progress in the States of Assam, Meghalaya and Arunachal Pradesh for the study of seismo-tectonic effects in the North Eastern Region of India which would also help the studies in connection with the project 'Survey of the seismicity and seismo-tectonics of the Anatolian-Zagros-Hindukush-Himalayan Ranges' as proposed in a Unesco meeting in Tehran (Iran) in 1974. The repeat observations would also provide us data to evaluate Crustal Movements in the region. A portion of levelling line from Guwahati to Dergaon was levelled during 1952–55 and repeated in 1973–76, covering part of Shillong Plateau. The results were statistically analysed. One of the bench-marks was found to be sinking with the rate of 1 cm per year.

The Shillong Plateau has a history of vertical uplift and is characterized by large position Isostatic anomalies accompanied with high seismicity. The Bouger anomalies in the region vary from -2 mgal at a bench-mark 35 km East of Shillong to -246 mgal at a bench-mark in North Lakhimpur (Assam Valley). Isostatic Anomalies (Airy, T=30 km) vary from +86 mgal to -155 mgal in these areas. The high seismicity in these areas indicate that the movements are continuing and that the Shillong plateau is still rising and has not reached isostatic equilibrium.

Further geodetic and geophysical observations have been planned for the study of seismic and seismotectonics effects in this area. These studies comprise of high precision levelling along the circuits Guwahati–Nangpoh–Shillong–Garampani–Newong–Guwahati and also Micro Gravity surveys. High precision levelling will also be carried out from Pasighat to North Lakhimpur for tectonic and geodynamics studies. Repeat geomagnetic observations will also be carried out in the Shillong Region in connection with establishment of Strong Motion Earthquake Instrument Array.

4.2 Loktak Hydro-electric Project Manipur

Project authorities had found some movement in the alignment of Penstock anchor blocks which had developed some cracks. Geodetic observations were planned.

Two Anchor Blocks and six saddles were selected for observations through simple triangles from a common base. Another Anchor block was chosen near the power house area.

Geodetic Triangulation and Precision Levelling observations were taken in 1977–78, 1978–79 and 1979–80. Computation of observations and interpretation of results are expected to be completed soon.

5. CENTRAL ZONE

5.1 Western India

There had been various views and interpretations on the structure, formations and tectonics of Aravali Range by the various Geodesists at different times.
The recent studies reveal that no support exists for the postulated Delhi-Haridwar extension of the Aravali's. Strong support is available for the extension of the Aravali's to the NW of Sambar Lake towards Lahore. It is also inferred from the trend of the Gravity anomalies that for isostatic equilibrium to be restored, the Aravalis would have to undergo epeirogen uplift. A levelling line of high precision, accompanied with geotraverse profiles (gravity and magnetic observations) have been established in the area during 1974-76 for subsequent repeat observations on another epoch to obtain quantitative evidence of epeirogen uplift and restoration of isostatic equilibrium.

5.2 Narmada Son

Central India

Repeat levelling observations were carried out along two profiles in the Narmada-Son Rift Zone during 1976-77. Earlier the levelling was established in 1936-37. No rise or subsidence was revealed. The Geotraverse profiles were also established during 1976-77 for detailed studies.

An estimate of the vertical crustal movements in Indo-gangetic plains has been attempted using the "Velocity-Surface Techniques". A surface describing the crustal velocities in the whole area has been computed by choosing a low degree of approximating polynomial connecting the crustal velocities of bench marks useful in geophysical investigations indicating the general trend of crustal activities.

6. SOUTHERN ZONE

6.1 Koyna Earthquake effect in Koyna Dam area

The Koyna Dam in Western side of the Deccan Plateau was constructed during 1962-67 for Hydro-electric Power generation. The area began to show wild seismicity since summer of 1962 with increase in the intensity. The earthquakes of magnitude 5.8 on 13.9.1967 and of magnitude 7.0 on 11.12.67 with their epicentres close to Koyanagar caused heavy loss of life and property.

The occurrence of these earthquakes could be either due to i) major earth- quakes of tectonic origin and tremors of lesser magnitude from reservoir settlement or ii) micro earthquakes due to leading of the lake and the major shocks to triggering action provided by water percolating downwards from the reservoir floor to release accumulated strain.

The above episodic drew attention of the scientists from all over the World and integrated geodetic and geophysical studies were set forth for ensuing safety to the engineering structure and further planning. These studies have also been utilized for RCM in the area.

The geodetic control in the area is available for the period prior to the occurrence of the 1967 earthquake. In 1965-66 points were established in the Koyna Dam area and this scheme was revised in 1968-69. Although the descriptions were found within the error of observations but the shift was systematic towards South-East by about 15 cm near the epicentre of the earthquake. The levelling in the area was available for epochs 1965-66, 1968-69 and 1971-72. These indicated that the area near Halwak about 5 km South of Koyna Dam and near the epicentre of earthquake had subsided by about 3 centimetres. At the Dam Axis no change was noticed.

New Schemes of horizontal control have been drawn for precise determination of Crustal Movements and pillars/stations established during 1975-76 with maximum load of the reservoir and at minimum load. No significant change was noticed due to loading of the reservoir.

6.2 Cochin Area

During the establishment of levelling network in the country, 2 bench-marks established as tidal reference bench-marks at the Cochin Port during 1887 were included in the First Level Net Adjustment in 1909. Since Cochin had about 6 years continuous and systematic tidal observations the local Mean Sea Level height of these bench-marks was included in the Indian Mean Sea Level Datum height.

While repeating the levelling network during 1953-54 for collecting data for the second level net, it was revealed that these bench-marks had sunk by about 0.1 m. Further repeat levelling during 1955-56, 1963, 1967-70, 1977-78 and 1978-79 are again showing a rising trend. This rise is of the order of 2 to 3 cm in 25 years. The Second Level Net adjustment in the country is nearing completion and all the previous levelling work will be revised to give the exact nature of the crustal movements in the area.

6.3 Koratti Basin

A fault line passes in the area. Some levelling bench-marks were available...
able near the area of 1884-85 levelling. Another levelling line was established in 1952-53. A fresh levelling line was established in 1975-76 in the close proximity and crossing the fault line. Although some variation was detected in some bench-marks from +5 cm to -14 in a period of 90 years but the variation was not systematic and conclusive. The repeat observations on another epoch may indicate the movements.

3 geotraverse profiles have also been established in the area with gravity and magnetic observations. The anomalies are being worked out for geophysical interpretations.

6.4 Pamban Pass area

The levelling of high precision carried out during 1955 revealed the subsidence of about 0.1 m in Pamban Pass area, as compared to 1880 levelling.

7. CONCLUDING REMARKS

From the above account it can be seen that the activity for study of Crustal Movements was initially incidental to geodetic levelling in the country. Subsequently areas of special interest were taken up for geodetic and allied geophysical studies.

More recently, the multipurpose developmental engineers projects are bringing for an expeditious study right from the pre-investigation stage.

This has resulted in studies in scattered isolated pockets.

Government of India is now taking up a major research project for the study of seismotectonics for the whole of the Himalayan Region. This will enable a systematic study for the whole region to be carried out, with one of its objectives to produce cartographic presentation of the pattern of crustal movements, and variations in gravity and geomagnetic anomalies.

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MONITORING OF CONTEMPORARY SEISMIC ACTIVITY FOR PROJECT SITES

ENREGISTREMENT DE L'ACTIVITE SISMIQUE CONTEMPORAINE SUR LES SITES DES PROJETS

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ABSTRACT

The need for monitoring of seismic activity as complimentary to engineering geological investigations for major projects like dams is already well understood. Indian Standards Institution brought out an Indian Standard 'Recommendations For Seismic Instrumentation For River Valley Projects' in 1969 (IS:4967-1968). A few projects followed these guidelines and installed network of seismological observatories. On examining the case history of one such network installed by Irrigation Department of U.P. Govt. in Tehri Garhwal Himalayas it is found that because of the limitations of the instruments installed in these observatories the major objectives of the investigations, viz. monitoring of contemporary seismic activity of the project site, are not fully achieved. The type and specifications of instruments and the number and spread of seismological stations that should be used in such local networks have been discussed.

ABSTRAIT

On sait déjà que l'enregistrement des activités sismiques est complémentaire aux études géologiques et de génie civil pour les projets de grande tels que dans barrages. L'Institut Indien des Standards a publié en 1969 (IS:4967-1968) des recommandations standards en ce qui concerne l'instrumentation sismique dans les travaux d'aménagement d'un bassin fluvial. Ces directives ont été suivies dans un certain nombre de réalisations et on a mis en place un réseau d'observatoires sismiques. Si l'on examine le cas d'un tel réseau mis en place par le Service d'Irrigation du gouvernement de l’État d'Uttar Pradesh, à Tehri Garhwal, dans les Himalayas, on constate qu'à cause des limitations des instruments installés dans ces observatoires, les principaux objectifs de ces investigations, c'est à dire, l'enregistrement des activités sismiques contemporaines sur le site du
projet, n'ont pas été entièrement atteints. On a discuté du type d'instruments avec les détails techniques; le nombre et la répartition des stations sismiques à mettre en service dans de tels réseaux locaux.

**INTRODUCTION**

Larger number of major river valley projects are located in highly seismic regions, nearer to areas which have experienced major earthquakes in the past. There are cases reported of damage to river valley projects by earthquakes. The occurrence of Magnitude 6.5 earthquake in December 1967 near Koyna dam site, hitherto considered moderately seismic, caused significant cracking of the dam which required a major repair and strengthening, besides the loss of life in the vicinity due to numerous collapsed houses (Berg et al., 1969, Krishma et al., 1969). Similar cases of damages are reported from other areas of the world, e.g. the damage to Hsinfengkiang dam in Peoples Republic of China (Bolt and Cloud, 1974). Therefore the likelihood of future damaging earthquakes must always be kept in mind in the construction of large river valley projects.

In order to study and provide surveillance of contemporary seismicity, locate active faults and monitor induced seismicity, it is essential that the proper instrumentation be installed in the project areas during investigation stage itself. In the absence of adequate instrumentation it is virtually impossible to systemati-

ically study the seismicity of the area to establish the extent to which local earthquakes were a consequence of the reservoir loading or they were part of a more general seismic pattern. Such a study also helps in providing clues to large earthquakes in future and also in assessing the danger of particular faults mapped geologically and thus is of immediate practical importance in determining the extent of foundation treatment.

The first major effort to provide seismic instrumentation for a river valley project in India perhaps followed the filling of Shivaji Sagar Lake, Koyna dam, Maharashtra in 1962 following which mild tremors were felt in the area by the end of 1963. During the same year the first seismological observatory was installed at Koyna and subsequently three other observatories were set up around the reservoir. All these four observatories were equipped with Wood-Anderson (magnification 1000) and one (vertical) component, short period, Benioff (magnification around 10 k) seismographs. Additional observatories were set up soon after December 11, 1967 earthquake. The details of instrumentation provided in these observatories are given in Table 1.

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<td>0.5</td>
<td>0.10</td>
<td>30:1</td>
</tr>
<tr>
<td>Kirnos (Koyna Quarry)</td>
<td>E</td>
<td>L</td>
<td>Z</td>
<td>-</td>
<td>013.75</td>
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</tbody>
</table>

\( T_O \) Seismometer period (Sec) T Transverse to Dam axis i.e. N 35°E
\( T_T \) Galvanometer period (Sec) L Longitudinal to Dam axis i.e. N 55°W
\( M_\text{E} \) Peak Magnification E East-West
\( W.A. \) Wood-Anderson N North-south
\( \text{EM(SP)} \) Short period, Electro- Z Vertical
Magnetic

*Commenced recording before the occurrence of Dec 11, 1967 earthquake"
period two other networks of seismological observatories, viz., the Beas and Kishau came into operation. The Beas network contained ten observatories having short period, high magnification, electromagnetic (Hagiwara type) seismographs. The network has been in effective operation since 1965. The network was particularly designed to provide data on seismic activity around Pong and Pardah project sites. The network for Kishau dam project in the region of Yamuna and Ganga valleys comprised of four observatories, viz. at Kishau, Narendra Nagar, Tehri and Rudra Prayag. Additional observatory was operated in the network at Kalawar for a very limited period. The instrumentation provided in these observatories has been given in Table 2. The network has been in effective operation since early seventies. Lately since the data from these observatories is being utilised for investigations related to Tehri dam project the network is now referred to as the Tehri network.

The awareness for seismological instrumentation for monitoring local activity was amply demonstrated by the initiative taken by the projects referred here. In order to facilitate effective planning and operation and also to bring in uniformity in the seismological instrumentation, Indian Standards Institution, New Delhi brought out an Indian Standard entitled "Recommendations For Seismic Instrumentation For River Valley Project" (IS:4967-1968) during 1969. The main features of the recommendations provided in the Standard are:

i) Five observatories for each project, one near the dam site (main observatory) and the others (subsidiary observatories) within an interstation spacing of 70 km.

ii) The main observatory would consist of (a) one complete set (two horizontal components and one vertical component) of short period (period of seismometer being 1.0 sec and that of galavanometer 0.1 sec), high magnification (being greater than 10000), electromagnetic seismographs and the recorder of these instruments will operate on a paper speed of not less than 60mm/min, (b) both the components of Standard Wood-Anderson type torsion seismograph (Period 0.8 sec, damping 80% critical and magnification 2800).

iii) The subsidiary observatory would consist of (a) one component (vertical) of short period, high magnification, electromagnetic seismograph and (b) both the components of Standard Wood-Anderson type torsion seismograph.

iv) The network should be operative five years in advance of the design stage of the project.

Over last one decade or so considerable new experience has
<table>
<thead>
<tr>
<th>Observatories</th>
<th>H</th>
<th>Site</th>
<th>Instrument</th>
<th>Component</th>
<th>T₀</th>
<th>T₉</th>
<th>h</th>
<th>h'</th>
<th>M (in K)</th>
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<td>Shales/ States</td>
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<td>1</td>
<td>1</td>
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<tr>
<td></td>
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<td></td>
<td></td>
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<tr>
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<td></td>
<td></td>
<td>May 1969</td>
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<td>Quartzite</td>
<td>EM(SP)</td>
<td>Z, N, E</td>
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<tr>
<td></td>
<td>30°33' N</td>
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<td></td>
<td></td>
<td>Feb 1972</td>
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<td></td>
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</tr>
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<td>Quartzite</td>
<td>Benioff</td>
<td>Z, N, E</td>
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<td></td>
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<td></td>
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<td></td>
<td>30°23' N</td>
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<td></td>
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<table>
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<td>h</td>
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<td>T₉</td>
<td>Peak Magnification</td>
</tr>
<tr>
<td>h'</td>
<td>Damping of Galvanometer</td>
<td>M</td>
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<tr>
<td>T₀</td>
<td>Seismometer period (Sec)</td>
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<td>Wood-Anderson</td>
</tr>
<tr>
<td>EM(SP)</td>
<td>Short period, Electro-Magnetic</td>
<td>Z</td>
<td>Vertical</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>North-South</td>
</tr>
</tbody>
</table>

*Date of commencement of recording*
been gained and significant developments have taken place in the seismological instrumentation. On the basis of these considerations and a review of the examples of instrumentation cited here will show that the efforts could be more beneficial if the instrumentation provided was not the same as provided in routine observatories for near earthquake recording (Agrawal, 1978). The case study of the Tehri network has been presented here and recommendations have been formulated for instrumentation which is more relevant to the usual objectives of seismological recording for engineering projects (Agrawal, 1978).

**TEHRI SEISMOMETRIC NETWORK**

Irrigation Department of Government of Uttar Pradesh, India proposes to construct a 260.5m tall rock fill dam across river Bhagirathi near Tehri town in Garhwal region (Fig.1). This project is an irrigation-cum-power project and will be the highest earth and rock

![Map showing major lineaments, rivers and earthquake epicentres](image)

**FIG.1.** MAP SHOWING MAJOR LINEAMENTS, RIVERS AND EARTHQUAKE EPICENTRES (LOCATED BY TEHRI NETWORK DURING JAN. 1975-MAR. 1980) IN GANAG AND YAMUNA VALLEY REGION OF GARHVAL HIMALAYAS

VIII.24
fill dam in the country. The area around Tehri is bounded by two major lineaments, viz. Main Central Thrust (MCT) and Main Boundary Fault (MBF) (Fig.1). The region had suffered violent ground shaking during 1905 Kangra earthquake whose auxiliary epicentre was close to Chakrata. The local geology and tectonics of the region has been mainly investigated by the various officers of Geological Survey of India (Shome and Kumar, 1979).

The rocks exposed in the vicinity of the Tehri dam site, are the phyllites of Chandpur series. Towards north and east of the dam site, these rocks are in contact with the Simla Slates. In certain sections, the phyllites are directly in contact with the younger dolomite and quartzite of the Garhwal group (Chamoli window). The contact between the Chandpur of Simla slates, on the one hand, and the rocks of the Garhwal group on the other, is marked by the Srinagar thrust, which can be considered to be a counterpart of the Krol thrust. The Srinagar thrust is having a strike continuation of over 100 km. The age of the thrust is not known but some consider it to be equivalent of Krol thrust in which case its age could be post middle Miocene. A few other well identified faults in the region are Gadolia Tear, Deul Tear, Tehri Tear, Marh Tear and Chamba Fault.

In view of the aforesaid geo-
tectonic activity of the region the installation of seismological network was very appropriate. Detailed specifications of instruments provided in the observatories are already mentioned in Table 2. Location of these observatories and response curves of the instruments installed in these observatories are shown in Fig. 1 and Fig. 2 respectively.

REVIEW OF DATA AND RESULTS

Because of some limitations in pooling data from all the four observatories presently operating in Ganga and Yamuna Valleys, the detailed analysis and interpretation of seismological data recorded by the three observatories in Ganga valley, viz. Narendra Nagar, Rudra Prayag and Tehri has been done. However, data from Kishau observatory was also taken for only selected events and the results finalised (EQ 80-14, 1980). These are given below:

1) Only 186 weeks of effective three station overlap recording could be had in the entire period of 274 weeks (Jan 75 – Mar 80). Table 3 shows the details of periods for which there were gaps in recording at the three observatories. The chief reason for the gaps is the non-availability of required power supply at the recording stations.

2) Over 1000 earthquakes (within 250 km) were recorded simultane-
TABLE 3 - Information on non-availability of Data from January 1975 to March 1980

<table>
<thead>
<tr>
<th>YEAR</th>
<th>STATION</th>
<th>&gt;30 days</th>
<th>&gt;10 days</th>
<th>&gt;5 days</th>
<th>&gt;1 day</th>
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<td>1</td>
<td>3</td>
<td>11</td>
<td>30</td>
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<td>THR</td>
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<tr>
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<td>1977</td>
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</tr>
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<td></td>
<td>RPG</td>
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<td>25</td>
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<tr>
<td>1979</td>
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<td>-</td>
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<td>1980</td>
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<td>RPG</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>5</td>
</tr>
</tbody>
</table>

ously at the three observatories. However this number is only about 7-10 percent of the total earthquakes recorded independently at each of the three observatories. Most of the earthquakes recorded at these observatories are teleseismic events.

3) Majority of the earthquakes recorded simultaneously at the three observatories in the network have magnitudes between 2 and 3. However, the network has recorded very few earthquakes of still smaller magnitudes.

4) Out of 1056 earthquakes recorded simultaneously at the three observatories only 377 could be located. However those lying with in about 200 km from Tehri seismological observatory are about only 160 in number.

5) The study of the temporal distribution of the earthquakes recorded at each observatory separately does not show any definitive trend. A clustering of earthquakes is noticed at distances of 40 and 110 kms from Tehri in its northern direction (Fig. 1)

6) The hypocentre locations obtained are by and large scattered in the region all around and those in the vicinity of MBF are fewer. Some of the epicentres located by the Network show association with a portion of MCT north-northeast of Tehri (Fig.1).
DISCUSSION

The various recommendations of IS:4967-1968 are now examined in the light of the experience gained in running the local networks for monitoring of contemporary seismic activity around major project sites particularly the Tehri network. The point wise discussion on more important aspects is as follows:

1) The recommendations regarding installation of five observatories for each project are not based on sound scientific reasoning. The location of earthquake involves determination of four unknown parameters, viz. latitude, longitude, focal depth and origin time. In addition, the seismic wave velocity must be known and if the same also has to be derived from the observations then a total of five unknowns are needed to be determined. Thus in order that these unknowns are determinable for each event, it should necessarily be recorded by five observatories. It may also be pointed out that the effect of horizontal layering in the crust on seismic wave velocity in such an application can not be ignored. For improved locations generally more observatories than the minimum requirement will be desirable. In addition complimentary experiments may be required for determination of velocity structure. Also many smaller magnitude events, which would generally constitute major part of the data, may not be recorded by all the observatories as is clearly borne out by the example of the Tehri seismic net. Therefore it will be desirable to have at least 8-10 observatories for each seismological network and seismometers should be located in bore holes for improving the signal to noise ratio and thereby increasing the resolution of the network as a whole. This arrangement would also permit detailed seismological investigations with regard to nature of source of earthquakes for selected events which are simultaneously registered at about eight stations.

Instead of instrumenting each project independently it is desirable to instrument the entire region suitably, where more than one projects are planned in future, as the earthquakes do not recognise the territories of various projects. For example, the area of interest in Ganga and Yamuna valleys will approximately cover 200 km (E-W) X 150 km (N-S) as shown in Fig. 1 and in order to permit detection of all earthquakes occurring in the region and having magnitude upto 1, the inter station spacing is required to be about 25-30 km. With this consideration and taking into account the terrain of the region and the level of seismic activity, the entire region can be effectively investigated by setting up about 25-30 observatories (Research Scheme, 1979). The data can be
utilised for all the projects planned in the region. If on the other hand taking IS recommendations for about one dozen major projects, as are planned in the region, about 60 seismological observatories should be setup. Thus the proposed consideration is not only scientifically sound but may in general yield economical results if overall development of the region is viewed.

iii) The installation of Wood-Anderson seismographs at each observatory serves very little purpose. Looking into Fig. 3 it becomes clear that this seismograph will record earthquakes of magnitude 2 and above (assuming that the minimum trace width and noise level discernible are of the order of 1 mm). Therefore it would be desirable to have Wood-Anderson seismographs at one or at the most two
Fig. 3. Trace amplitudes for earthquakes having magnitudes 1(---) and 3(—) at various distances for seismographs having different magnifications.

Observatories for the purpose of comparison of magnitudes determined with the help of other instruments.

The characteristics of high magnification and short period electromagnetic seismographs are such that they would respond most favourably to teleseismic events having periods of the order of 1 sec (Fig. 2). This is obvious from the results of Tehri network.
also. However, the local earthquakes are characterised by their high frequencies of the order of 10 Hz or so. Naturally, most of these local earthquakes may not be picked by these instruments. For monitoring the local earthquakes, it is essential to use systems which employ amplifiers to increase the magnification over a very wide frequency range. When such a system was employed for short term seismic sampling in Tehri region, many local events were recorded (Kumar et al., 1980) whereas these were not registered by the existing network.

There is no scientific justification for recommendations restricting the 3 components of seismometers to only one observatory near the proposed site. The provision for having the three components of seismometers should depend upon the availability of funds and if there is enough money available, then more number of stations could be provided with three components of seismometers. If only one observatory can be provided with a set of three components of seismometers, the decision as to which one should be guided by the experience of the effective past recording.

Since the low level seismic activity has migratory character, provision should be made of employing a few (about five in case of a wider area like Ganga and Yamuna valleys) portable instruments in the network. In addition to these portable ones, the monitoring should be carried out by a network of telemetered array with central recording and common time base facility. This would overcome the problem of non-availability of adequate power supply at remote sites in addition to providing better time accuracy.

iii) Since the purpose of these observatories is to register local earthquakes, which have very high frequency (10 Hz or more), the speed of recorders employed for the purpose should be a few times larger than the recommended ones. The speed should normally be 120 mm/min or 240 mm/min.

iv) The network should be operative in the region immediately after the finalisation of the programme of its development and therefore as many years in advance as may be possible, instead of only five years, of construction of the project.

RECOMMENDATIONS

i) For deciding the optimum number of observatories to be installed in a particular region due consideration should be given to the objectives of the study, terrain and noise level.

ii) The instrumentation should consist of a velocity transducer coupled with a suitably matched high gain amplifier. Transducer-cum-amplifier should have maximum response $10^5$ or $10^6$ Volts/cm in
the frequency range of 0.1 to 100 Hz. Provision should be made for low and high pass filters in various combinations to allow improved recording at sites with noise in some selected frequency bands.

iii) Higher speed (120 or 240 mm/minute) recording should be done for better time resolution as the local earthquakes have high frequency.

iv) For improving the signal to noise ratio, the seismometers should be installed in a pit or borehole.

v) Telemetered array with central recording and common time base facility should be preferred for overcoming the problems of difficult terrain and nonavailability of power supply at remote sites, etc.

vi) A few independent portable micro-earthquake recorder units should also remain available for providing adequate coverage to some possible gaps in the existing networks created temporarily from time to time by the migratory character of the low level activity in some parts of the region.

ACKNOWLEDGEMENTS

Sri Pawan Kumar, Research Associate and Shri Sunil Kumar, Research Assistant, Department of Earthquake Engineering assisted in drawing the diagrams. Thanks are also due to Professor and Head, Department of Earthquake Engineering, University of Roorkee, for providing the necessary facilities.

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ABSTRAIT

L'île Grand Nicobar est située le plus au sud de la chaîne des îles de l'Andaman et Nicobar à l'Inde. Un tremblement de terre de la magnitude Richter 6.3 (Ms) s'est arrivé à côté orient de l'île le 20 Janvier, 1982, à 4 h 25 GMT et a provoqué une panique et un dégât considérable. Départe ment de la technique d'ingénieur de tremblement de terre de l'Université de Roorkee, avait organisé une investigation pratique de la région effectuée. Quelques dégâts typiques sont présentés dans cet article. Les données concernant l'affaiblissement de terre et des autres effets sismiques ont été présentées et la carte isosismique préparée. La location de l'activité sismique continue était évalué selon l'enregistrement à trois emplacements avec une seule station portative. Le modèle pour migration d'épicentre de l'activité du Janvier 1982 à été préparé.

ABSTRACT

Great Nicobar is the southern most island of the Andaman and Nicobar chain of islands. An earthquake of Richter magnitude of 6.3 (Ms) occurred on the east coast of the island on January 20, 1982 at 4h 25m G.M.T. and caused considerable panic and damage. Department of Earthquake Engineering, University of Roorkee, organised a field survey of the affected area. Some of the typical damage has been described here. Data on ground failure and other earthquake effects has been presented and the isoseismal map prepared. The location of continuing aftershock activity was estimated on the basis of recording at three locations with a single portable station. The pattern of epicentre migration of January 82 activity has been given.
INTRODUCTION

Great Nicobar is the southernmost island of the Andaman and Nicobar chain of islands, about 510 km from Port Blair and is largest in size amongst Nicobar group which has 19 islands (Fig. 1). Its southernmost tip namely the Pygmalion Point is only 145 km from Sumatra. The area of this island is 850 sq. km. The local inhabitants are Shompens and Nicobarese both being tribes of Mongoloid origin. The number of Shompens, a very shy and reserve tribe, is about 200. Due to the strategic importance of the island a program of resettlement of exservicemen was undertaken by Government of India in 1969. Now there are over 300 exservicemen families from nine states spread over seven villages along the east coast to the south of Campbell Bay which are the primary component of the island’s population of about 6000.

An earthquake of Richter magnitude of 6.3 (Ms) shock the island on January 20, 1982. The parameters of the main shock and some of the bigger aftershocks as taken from the preliminary determination of epicentres of USGS are in Table 1.

The epicentres of January activity are plotted in Fig. 2 and show the pattern of epicentre migration.

A detailed damage survey including collection of data on ground failure and other earthquake effects and recording of the continued seismic activity has been carried out by the Department of Earthquake Engineering, University of Roorkee. An isoseismal map has also been prepared and presented.

GREAT NICOBAR

The district headquarters for the Nicobar district is located at Kar Nicobar and has two subdivisions, namely Kar Nicobar and Nancowry. An Assistant Commissioner (Rehabilitation) is located at Campbell Bay. There are only two Higher Secondary Schools in Nicobar district and one of these is at Campbell Bay. In addition, there is one Senior Basic and five Primary Schools at Great Nicobar. Andaman Public Works Department has a subdivision at Campbell Bay. There are total three Hospitals in the district of which one is at Campbell Bay and is good enough for about ten patients. In addition to this there is a veterinary hospital.

There are two major roads at Great Nicobar connecting Campbell Bay to Kopenheat (E-W Road) and to Shastri Nagar (N-S Road). These are shown in Figure 2 and are constructed by Border Roads Organisation, Yatrik who now even look after other construction for the administration. The Andaman Harbour Works has a jetty at Campbell Bay. T.S.S. Yarewa and M.V. Onge are two ships which ply between Port Blair and Campbell Bay taking about one week for return journey. In addition to Great Nicobar-Katchal and Camorta
have jettys. An Indian Air Force flight also links Port Blair and Car Nicobar.

The island has very thick forest and has fertile land. Coconuts, betelnuts, cashew, banana, papia and various species (clove, pepper, cinnamon) grow on the island. A major rubber plantation project at Katchal is in progress. The climate is tropical, humid, more uniform and the maximum temperature annually change from 24 to 35°C except for heavy rainfall which does not occur for about two to three months (January to March). The island particularly the coastal areas are
TABLE I - PARAMETERS OF EARTHQUAKES DURING JANUARY 20, 1982 IN GREAT NICOBAR ISLAND REGION

<table>
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<th>DATE</th>
<th>ORIGIN TIME</th>
<th>EPICENTRES</th>
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<th>MAGNITUDE</th>
<th>SHOWN IN MAP AS NO.</th>
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<td>LONG. (E°)</td>
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<td>5.6 6.3</td>
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<tr>
<td>20.1.82</td>
<td>07 09 18.8</td>
<td>7.122</td>
<td>93.970</td>
<td>39</td>
<td>5.7 6.2</td>
</tr>
<tr>
<td>20.1.82</td>
<td>22 11 42.3</td>
<td>7.341</td>
<td>94.02</td>
<td>36</td>
<td>5.0 4.6</td>
</tr>
<tr>
<td>22.1.82</td>
<td>18 24 38.0</td>
<td>7.08</td>
<td>94.048</td>
<td>33</td>
<td>4.7 -</td>
</tr>
<tr>
<td>25.1.82</td>
<td>10 27 09.6</td>
<td>6.888</td>
<td>94.074</td>
<td>33</td>
<td>5.1 4.8</td>
</tr>
</tbody>
</table>

+ MS Pasadena = 6.0

infected with malaria. There is no wild life in forests. However, Crab eating monkeys are found in coastal area and some wild boars.

GEOLOGY AND SEISMICITY

The Great Nicobar falls on the non volcanic island arc in the Bay of Bengal to the South of the ten degree channel. It has three rivers namely, Galathia, Alexandria and Dagma. Unlike the mainland rivers these have practically no gradient and are largely tidal rivers. The terrain is hilly and the highest peak the Mount Thullier is 642 m high. Due to the remote location of Great Nicobar in particular and other Andaman and Nicobar islands in general the region has not received adequate attention for geological and geophysical studies although on north (Burma) and south (Sumatra) both petroleum and gas are found and there is a strong possibility of their occurrence in the region.

The Great Nicobar is one such single island which has the complete stratigraphic sequence of the Andaman and Nicobar island arc, i.e., rocks of late Cretaceous to Recent and Sub recent exposed at some or the other locality. The phyllites exposed on the island belong to the Prolob group. The ultrabasic and basic rocks designated as ophiolites belong to the Serpentine group (late Cretaceous to Eocene) are exposed on the east coast and are used as building material. The most dominant rock type on the island is Andaman flysh (grey sandstones, slates and shales) belonging to the Fort Blair Group. The sediments of Archipelago group on the island are silty mud stones, clays and lime stones rich in microfossils. There are recent coral bed and fringing reef formations.

The Andaman and Nicobar islands have undergone four or more orogenic movements. The most recent one was in the late Cenozoic. If at all any the island presently shows some evidences of a very small and
gradual uplift.

The Great Nicobar lies on the southern part of the Great Alpide-Himalayan Seismic belt and has been known for the seismic activity. The earthquakes of Richter magnitude up to 8 have occurred in the region. The epicentres of 1941 and 1955 earthquakes of magnitude 8.1 and 7.0 respectively are shown in map in Fig. 1. The activity near Great Nicobar has been generally below Richter magnitude 7 (more often close to 6). The epicentres of all the earthquakes in the neighbourhood of Great Nicobar as reported during 1920 to 1980 are given in Table II and have been shown in the map in Fig. 2. Generally the locations shown for these earthquakes are in the sea but reports of any tidal waves are not available. The past data also tend to show that moderate to major earthquakes in the Andaman and Nicobar islands have generally been characterised by aftershocks sequences whereas foreshocks have not been common.

**DAMAGE SURVEY**

The typical damage to civil engineering structures, roads and ground has been studied in detail only typical damage has been described here in the paper.

**Civil Engineering Structures:**

The most important structure which suffered damage is the jetty at Campbell Bay. The junction between jetty and ground got separated by about 20 cm. There were pillars on either side of the jetty at this place. The one on the right has been completely broken apart (Fig. 3) whereas the one on the left side remained in position although completely detached from the jetty. The ripping of the concrete and exposing of the reinforcement below this location had been caused. This could be found at other places also. Almost all construction joints got opened up to 5 cm and even suffered rotational displacement causing lateral and vertical shift of adjacent spans up to 7.5 and 6 cm respectively (Fig. 4). A closer examination of the forward span showed its larger vibration with respect to the inner span and tilting of piles. The relative displacement of berthing section and the approach has been about 15 cm. Close to the inner corner of this L-junction is a concrete lamp post which has been broken at its mid height (Fig. 5).

A large number of buildings got damaged. The buildings of the veterinary hospital in R.C.C. and hollow block construction had extensive damage. Seen in Figure 10 is a broken lintel. In another block the entire window along with the lintel has fallen apart (Fig. 6). The new school building under construction at Campbell Bay is completely razed to the ground leaving door frames and reinforcement bars standing bare. Partially standing construction in one corner only is seen in Figure 7. The roof of the cement shed at Yatrik headquarters has collapsed (Fig. 8). A shed at the Rehabilitation Department's
### Table II - List of Earthquakes in the Neighbourhood of Great Nicobar Island (1920-1980)

<table>
<thead>
<tr>
<th>Date</th>
<th>Origin Time H M S</th>
<th>Epicentre 0N 0E</th>
<th>Magnitude</th>
<th>Shown in Map as No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>24 Jan. 1924</td>
<td>18 34 42</td>
<td>7 94</td>
<td>5-6</td>
<td>7</td>
</tr>
<tr>
<td>17 May 1955</td>
<td>14 49 49</td>
<td>6.7 93.7</td>
<td>7.0</td>
<td>8</td>
</tr>
<tr>
<td>7 Sept. 1956</td>
<td>09 03 39</td>
<td>7.1 94.1</td>
<td>6</td>
<td>9</td>
</tr>
<tr>
<td>18 July 1960</td>
<td>00 54 07</td>
<td>7.0 94.0</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>6 Sept. 1964</td>
<td>18 57 20.4</td>
<td>7.1 93.7</td>
<td>5.2</td>
<td>11</td>
</tr>
<tr>
<td>8 Feb. 1970</td>
<td>00 08 07.3</td>
<td>6.7 93.5</td>
<td>5.2</td>
<td>12</td>
</tr>
<tr>
<td>9 Jun. 1971</td>
<td>07 47 45.2</td>
<td>6.8 94.0</td>
<td>-</td>
<td>13</td>
</tr>
<tr>
<td>25 Mar. 1976</td>
<td>08 16 28.0</td>
<td>7.1 94.1</td>
<td>5.4</td>
<td>14</td>
</tr>
</tbody>
</table>

Establishment has been tilted (Fig. 9). Several ordinary huts constructed by the settlers had also tilted. The damage to the floor of the buildings was extensive. Separation of columns from the walls along their joint was conspicuous (Fig. 11). The chimneys in the veterinary hospital staff quarters and others suffered extensive damage. The two-storey residences of marine engineers had very little damage.

Approach to a road bridge on Maggar Nallah got separated and some of its timber piles had been tilted.

### Ground Failure and Land Slide:

The ground failure and fissuring could be seen at a number of locations and a detailed mapping has been done for which the results will be separately reported. The ground particularly the road failure was seen up to about 12 to 15 km from Campbell Bay on both the E-W and N-S roads. The damage to road and floor of huts on E-W road were mainly due to the failure of side land slopes which resulted in roughly longitudinal fissuring on the road. A small land slide also occurred on E-W road at 27 km (Fig. 13).

The damage on N-S road in addition to the slope failure was also related to the failure along the coast line. At several sandy coast lines namely near Campbell Bay jetty, Fishermen Colony and Maggar Nallah well developed ground failure occurred showing relationship to the orientation of the coast line itself i.e., the fissures running roughly parallel to coast line (Figure 14).

In addition to the ground damage along the roads particular mention need to be made for the following ground fissuring as observed in Campbell Bay itself.

A dominantly N40°E running ground fissuring through the Campbell Bay market which could be mapped for a distance of 300 to 400 m. The Rehabilitation Department, guest house and the school buildings were located in this zone. Some informers even mentioned extension of
this fissuring across the hills as well but the same could not be verified. The detailed mapping of this fissure near guest house where the opening was up to 2 cm and weak twigs could easily be inserted to a depth of 75 cm or more.

The other prominent fissuring with by and large the same orientation was observable to about 200 m at the veterinary hospital complex. At some spots on this as well as the earlier zone of ground fissuring it was difficult to exclude the role of contraction in the ground fissuring as the pattern of fissures was typical of those due to contraction. However, their large depths and fresh widening was certainly caused by the earthquake shaking. Some settlers were telling their observation that the thermal contraction cracks were getting opened wider even prior to the earthquake actually took place.

At the Yatrik headquarters also a more or less linear ground fissuring was observable to about 100 m in N 10°E. Figure 12 shows a portion of this failure. The other directional effects of earthquake ground shaking were compatible with this direction of fissuring.

In conclusion it can be said that ground failure observed was primarily due to slope failure, differential settlement of loose soil and thermal contraction but the earthquake shaking did play a significant roll as trigger or enhanced the effects.

OTHER EARTHQUAKE EFFECTS

A number of other observations as reported to the team or observed by them are listed here for future reference:

1. There were no tidal waves and the general impression of most people along the coast is that the sea has receded due to earthquake in particular and may be receding gradually due to other reasons over the last couple of years.

2. Cattle reported rest-less and flocked together on road crossings.

3. Couple of dead rodents detected in lawns, and open ground when they started giving foul smell after few day of the earthquake.

4. Sound of cannon firing and not that of rumbling.

5. At Govindnagar people reported increased and muddy water from a spring.

6. Liquefaction of sandy soil at Campbell Bay:

i) Opposite Assistant Commissioner's (Rehabilitation) office where fine whitish and bluish material also came out and got deposited around openings through which water was reported to have come out temporarily.

ii) At fishermen colony where it was reported that there was
temporarily increased water in some spots where generally water stagnated but after the earthquake those have become dry. Some people reported smell of sulphur as well.

iii) At Campbell Bay jetty and Maggar Nallah causing tilting of deep piles.

7. Overturning of two steel Almirahs (Godrej) in the Syndicate Bank.

8. Overturning of a wooden stool (100x50x45cm and 12kg in weigh) and sliding of a pen stand by 20 cm on office table in N 150° in the office of the Commandant Yatrik.

9. A heavy (may be 200kg) cash safe on a flexible wooden stool (45x45x60 cm) caused swing and permanent set by about 15° to stool. The overturning was obstructed by support to the safe on its side by another taller steel Almirah.

10. Dislodging of two large oil tankers of 2700 Lt. capacity from specially made supports at the Rehabilitation Department's establishment at Campbell Bay.

11. At the Police Station a pair of High Frequency receivers each weighing 70 kg mounted (slidable) on a steel rack one over the other placed on a wooden table (150x90x75 cm) were pushed out.

12. A 50 kg 3 KVA diesel generator placed on a pair of 9 cm high wooden blocks over turned at the police station.

On the basis of these observation computation of the ground acceleration can be made separately and compared to arrive at an estimate of the same. However, no such computations have so far been done but it is estimated that the ground acceleration was certainly around 15 percent g.

**ISOSEISMAL MAP**

On the basis of the observations of damage caused by the January 20, 1982 earthquake intensity values have been assigned to various localities and a map showing isoseismal lines has been prepared and given in Figure 2. The intensity estimates given here are in general on conservative side. It should be noted that the observations were obtained mainly from N-S and E-W roads and at other places where the observations were not made the lines are shown broken. Efforts have been made to correct the bias on the damage observations of the variation in type and quality of construction, population density, the foundation conditions, etc. It may be noted that the January 20, 1982 earthquake epicentre location marked with No.1 which became available after the preparation of the map fits in satisfactorily with the analysis of observations. The close spacing of different isoseismal is also compatible with the focal
depth of 28 km. However, on the basis of off shore location as given by USGS the tidal wave were expected to be observed which has not been the case. A possible explanation is proposed to be discussed in the detail report on ground failure.

MICROEARTHQUAKE RECORDING

One short period, high gain, high frequency portable earthquake recording instrument was taken to the area and was mainly installed at Campbell Bay guest house. For short duration it was operated at two other locations, namely 27 km on E-W road and 39 km of N-S road. Presuming that the source of activity over the period of observation remained stationary and in other words considering recording at the three places as simultaneous the location of epicentre was obtained and the same has been shown as the probable epicentre in figure 2. The focal depth estimate so obtained is 18 km. This location is quite a bit different than the one given by USGS for the January 20, 1982 event. It is interesting to note that the data for 5 aftershocks given by USGS showed a systematic migration of the activity first to the north and then return to further south of the original epicentre. Perhaps, the epicentre of the current activity is not the same as that of the main event. However, the three events which were felt as well as recorded during the stay produced a similar variation of ground shaking as was observed during the January 20, 1982 event. It may also be pointed out here that the seismological network of station as available should provide better control on the determination of latitude than of longitude where as in the actual observations the locations of current activity show a greater scatter in the latitude and thereby indicating a definite north-south migration of the activity.

CONCLUSIONS

1. The earthquake resistant design and construction practices using local materials have to be developed for the region particularly for buildings. This is unlike most other seismic regions in the country where people living for generations have by themselves used suitable technology, like, Ikra construction in Assam. Many lessons can however be learnt even from the simple dwellings of Nicobares and Shompens.

2. The Campbell Bay jetty needs immediate repairs and strengthening at least at those points where piles have tilted or superstructure is damaged.

3. In view of the expected development of Great Nicobar due to its strategic importance, prospects of finding oil and other economic minerals, agricultural potential and possible economic cultivation of cash
crops it is recommended that a local network of seismological stations should be operated in the region for collecting detailed seismic data.

ACKNOWLEDGEMENTS

The Survey was assisted by Dr. A.K. Jain, Reader, Department of Earth Sciences and Sri Dinesh Chandra, S.S.A., Department of Earthquake Engineering, University of Roorkee.
Fig. 3 - Right Side Pillar on Campbell Bay Jetty is Broken Apart

Fig. 4 - Broken Parapet of Jetty Shows Lateral and Vertical Displacement of 60 cm respectively

Fig. 5 - A Lamp Post on Jetty Broken at its Mid Height

Fig. 6 - Collapse of a Large Window Along with the Lintel

Fig. 7 - Damage to a Hollow-block Building Under Construction

Fig. 8 - Collapsed Roof of a Cement Shed at Yatrik Headquarters

Fig. 9 - A Vehicle Shed at the Rehabilitation Department Tilted by 15°
Fig. 10 - Lintel Damage in a Veterinary Hospital Building

Fig. 11 - Vertical Separation of Columns and Wall

Fig. 12 - Linear Ground Failure near Yatrik Headquarters

Fig. 13 - A Landslide at 27 km E-W Road

The major help enabling us to reach the Campbell Bay came from the Fortress Commander, Naval Establishment, Port Blair. The prompt cooperation of the Andaman and Nicobar Administration enabled expeditious survey.

The Department of Earthquake Engineering, University of Roorkee have funded the survey.

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Fig. 14 - Ground Fissuring near Maggar Nallah Bridge


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SEISMIC ENGINEERING GEOLOGY EVALUATION OF SHENGYANG-FUSHUN AREA, CHINA
EVALUATION DE LA GÉOLOGIE DE L'INGÉNIEUR SISMIQUE DE LA RÉGION DE SHENGYANG-FUSHUN (CHINE)

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ABSTRACT

The Shenyang-Fushun area is located within the area showing an intensity scale 7 in the regional planning map of National Earthquake Intensity. The problems of seismic engineering geology have to be resolved immediately for the city planning and site selection in this area. This paper consists of three parts: 1. The background of seismic geology and the prediction of riskful area; 2. The engineering geologic analysis of site for strong seismic effect; 3. Main seismic engineering geologic problems and their evaluation. This paper demonstrates that the area is fraught with seismic risk of 5-6 degree and with three risky positions. By the study of seismic engineering geologic condition of the site, using the theory and method of response analysis of ground movement, the basic parameter value of ground movement of this area and contributing regularity of dynamic characteristics at the site are determined. Finally, the evaluation of main problem of seismic engineering geology and site condition has been done, with this foundation, the future effect of seismic calamity for both Shenyang and Fushun cities has been predicted.

ABSTRACT

La région de Shenyang-Fushun se situe entre la région montrant une échelle de l'intensité dans la zone sismique à l'intensité VII, classée par l'État. Il est urgent de résoudre les problèmes relatifs immédiatement à la géologie de l'ingénieur sismique pour l'urbanisme et le choix des sites dans la région considérée. Cet article comprend trois parties: 1. Le contexte de la géologie sismique et la prévision des zones dangereuses; 2. L'analyse géotechnique des effets sismiques dans les sites; 3. Les problèmes principaux de la géologie de l'ingénieur sismique et leur évaluation.

L'article démontre qu'un séisme dangereux de magnitude 5 à 6 se prépare dans la région et qu'il existe trois places dangereuses.

Après des études des conditions géotechniques sismiques des sites, les valeurs des paramètres fondamentaux du mouvement de terrain rocheux et la loi de répartition des caractéristiques dynamiques des sites dans cette région ont été déterminées à partir de la théorie et de la méthode analytique du mouvement de terrain. On finit par évaluer les principaux problèmes de la géologie de l'ingénieur sismique et les conditions des sites. Et sur cette base, une prévision des effets de risques sismiques pouvant avoir lieu à l'avenir dans les deux villes, Shenyang et Fushun à été faite.

INTRODUCTION

Shenyang-Fushun area lies in the heart of Liaoning Province. It is situated in the northern part of the strong seismic region of Tanchen-Haichen in East-
ern China. In order to plan the layout of cities rationally and to make the choice of sites more reasonable, from the prognosis of seismically dangerous nature of the region, this paper deals with the basic laws and characteristics of its ground movement under strong seismic processes on the basis of engineering geology of rock and soil in situ and by evaluation of seismic engineering geology.

A. The background of seismic geology and the prognosis of dangerous seismic shows of the region

Based on the study of regional geology and tectonics of the Shenyang-Fushun area, its seismic geological condition can be generalized as follows:

(A) The area is located along the Yingkou-Kaiyuan fault zone (Shenyang-Kaiyuan fault and Damintun fault in this area) the northern extension of the Tan-Lu active fault zone in Eastern China and having a trend varying from northeast-east like the Hun River fault zone. Further, there are also developed east-west and north-west holding faults (Fig. 1). They respectively belong to the Neocathaysian and cathaysian latitudinal tectonic systems and represent the northwest structural constituent.

![Fig. 1. Seismic Geological Map of Shenyang-Fushun Area; 1. active fault; 2. Neocathaysian fault; 3. Cathaysian fault; 4. east-west tectonic fault; 5. north-west fault; 6. the isopach of the quaternary sediment (unit meter); 7. the epicenter of historical earthquake; 8. signs for each fault: 1) Hun River fault; 2) Damintun fault 3) Shenyang-Kaiyuan fault; 4) Shenyang-Nluxintai fault; 6) Ximin-Danxinggou fault; 7) Daliutun-Yuhou fault; 8) Beiling-Yaojiaogianhu fault.](image)

In recent times Yingkou-Kaiyuan fault within the Neocathaysian system is notable for inheriting seismic activity. It runs along a mountainous stretch to the east of Shenyang and on a plain to the west of it. The city district is just on the alluvial and pluvial fan of Hun River in the sloping plain in front of the mountain. But Fushun area is in the rising part of the mountain land to the east. Only along the valley of Hun River the Quaternary System 10 to 15 meters thick has developed.

(B) As regards seismic activity, it is recorded in history that an earthquake of magnitude 5.5 took place in 1765 in Shenyang City. In the past 600 years, over 100 earthquakes have occurred and been felt in Jin County-Shenyang-Kaiyuan regions. Among them, damages of earthquakes amounted to ten. But on the fault zone of Hun River only several small earthquakes have occurred. It can be seen from the seismic geological map (Fig. 1) that the seismic activities were most closely related to north-north-east fault. And earthquakes occurred mostly at the convergent position of several faults.

(C) From the analyses of geophysical data on the crustal structure in the area, it is seen that the crust of the earth is 30 to 32 kilometers thick in the western plain and 35 to 36 kilometers thick in the eastern and the western sides of the crust are nonhomogeneous in depth in the area and that this difference in crustal thickness is controlled by the Yingkou-Kaiyuan fault (Fig. 2).

![Fig. 2 The depth of Moho-Kangla boundaries in South Liaoning](image)

1. The isoline of the depth of Moho boundary
2. the isoline of the depth of Kangla boundary

(D) The latest tectonic stress field in Shenyang-Fushun area exerts direct control of the seismic activity there. From
data of earth stress survey in recent years in the Liaoning and Jilin Provinces (Table 1) and computed solutions on seismic focus on a large scale in the neighboring area, it is proved that the direction of the main compressive stress of the recent regional tectonic stress field is north-east to south-west.

<table>
<thead>
<tr>
<th>Surveyed Line</th>
<th>Place</th>
<th>Maximal Stress Value (kg/cm²)</th>
<th>Trend Orientation</th>
<th>Surveying unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>1925-1931</td>
<td>Fushan</td>
<td>96.5</td>
<td>44.87</td>
<td>42.85</td>
</tr>
<tr>
<td></td>
<td>Wuji</td>
<td>75.0</td>
<td>42.01</td>
<td>43.75</td>
</tr>
<tr>
<td>1974.4-5</td>
<td>Beilin</td>
<td>107.75</td>
<td>56.16</td>
<td>42.75</td>
</tr>
<tr>
<td></td>
<td>Shenyang</td>
<td>103.47</td>
<td>59.24</td>
<td>44.37</td>
</tr>
<tr>
<td>1975.7</td>
<td>Beilin</td>
<td>107.40</td>
<td>53.42</td>
<td>42.35</td>
</tr>
<tr>
<td></td>
<td>Shenyang</td>
<td>103.47</td>
<td>59.24</td>
<td>44.37</td>
</tr>
</tbody>
</table>

Table 1. Earth Stress Survey of recent Years in Liaoning and Jilin Provinces.

Further, in order to determine the dangerous areas susceptible to earthquakes and determine the trend of such shocks this paper has adopted analogue calculation of mathematics and mechanics and done the analysis of limited unit of regular net squares so that the distribution of Stress within a depth of 15 kilometers can be ascertained. Finally, a comprehensive resultant plan (Fig. 3) has been drawn. Five stress concentrated regions have been determined in the area. Analysis of state of stress along the fault zone has been determined by conversion matrix of the straight stress and the shearing stress of the fault plane. The analysis of state of stress can be done in accordance with the rule of shearing rupture. According to this, the analysis of numerical value is carried out for Shengyang-Kaiyuan fault, Damintun fault and Hun River fault closely relating to the activity of earthquake. The result shows that Wangbin-East Xinchengzi on Shenyang-Kaiyuan fault, Yuhong nearby on Damintun fault and Suijatun-Guchengzi on Hunhe fault, are most probably sheared.

The trend and intensity of a dangerous earthquake occurring along the fault can be analysed as follows:

1. Based on the relation between magnitude and time in North China, the area falls under the fourth active period recorded since 1740. It will still remain in it if the fourth active period is 60 years.

According to the relation between magnitude and frequency of the area, the value b in recent years can be worked out. And it is most likely that the earthquake of magnitude 5 to 6 may break out here. The earthquake of M > 5 has not taken place in Shenyang and places nearby for about 200 years. The prognostics of pregnancy of earthquake and of energy accumulation is worth investigating.

2. From the formula $M = 3.3 \times 2.1 \log L$ of magnitude calculated by intensity (L represents the length of active fault in kilometer ) it is also worked out that there exists the danger of a strong earthquake shock of magnitude 5 to 6 in 100 years.

Based on the analyses above, three dangerous regions of strong earthquakes of M 5 and two dangerous regions of weak earthquakes of M 5 have been determined in a preliminary manner (Table 2).

![Fig. 3. Comprehensive plan of the calculation of the limited unit of present stress field in Shenyang-Fushun area.](image-url)
B. Analysis of strong earthquake effects on engineering geology of site:

The difference in the seismic effect on the various factors of the site in the area will be further analyzed on the basis of the study of dangerous seismic regions stated above.

(A) The geological characteristics of the site in Shengyang-Fushun area:

1. The characteristics of the distribution of lithofacies: from east to west there exists bed and alluvial and fluvial beds, in front of the mountains alluvial beds of the plain and the alluvial beds of the Hun Valley. The lithofacies change apparently in the horizontal and vertical directions (Fig. 4, Tab. 3).

The city district of Shengyang lies on the alluvial and pluvial fan. The present valley of Hun River is developed to the south of this region. It is made up of sand, gravel and medium and fine sand beds, mud and soil are seen partially. The lithofacies is mainly alluvium and the glacial drift and glacioaqueous sedimentary beds are buried at depth with a loess-like soil on the top (Fig. 5).

2. Deep fault and topography of bed rock: from Figure 1, it can be seen that the fault is wholly hidden in the plains area due to the overlying Quaternary sediments around Shengyang. It is verified from the drilling data of coal field and hydrogeological data that the faults 1 to 8 area all fault zones composed of several faults. Each has a width of over ten meters to tens of meters, and deeply cuts into the earth's crust for depths of a kilometer to tens of kilometers.

The ground surface appears high in the east and low in the west, and the distance in between is only 10 to 20 meters around Shengyang. Below the overburden there is a 'hidden mountain or hill' whose main body is connected with the mountainous terrain to the east. Buried valley of loose sediment of 100 to 150 meters thick lies to its south, west and northwest. Obviously, this kind of fault zone and topography of buried bed rock directly influence the differences in strong seismic effect.
3. The depth of ground water is getting shallower from east to west in Shenyang area. It is around 15 to 20 meters below the ground in the city district, and over 30 meters in few pumping places. In the Hun Valley floor it is only 3 to 5 meters.

(B) The wave velocity of foundation soil in the site and elastic shearing modulus:

The wave velocity and its shearing modulus are related to the type of ground at the site, the size of grain, the relative density of the ground and the age and nature of the formations at the site.

<table>
<thead>
<tr>
<th>Geological Age</th>
<th>Lithofacies</th>
<th>Maximum Thickness (m)</th>
<th>Distribution Range</th>
<th>Velocity of Transverse Wave (Vp/Vs)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Holocene</td>
<td>3. artificial soil</td>
<td>3 - 5</td>
<td>city district the banks of Hun River flood bench of Hun River, the first terrace</td>
<td>appeared on the earth's surface not widely distributed depths: 0-20(m)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. clay sand (or mud)</td>
<td>10-20</td>
<td>the first terrace of Hun River plain</td>
<td>mostly distributed in the shallow below the earth's surface depths: 0-20(m)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1. sand &amp; gravel</td>
<td>6 - 8</td>
<td>down part of city district down part of city district</td>
<td>partly seen in the shallow, mostly buried in the depth depth: 0-50(m)</td>
<td></td>
</tr>
<tr>
<td>Lower Pleistocene</td>
<td>3. clayey soil clay sand &amp; loess-like soil</td>
<td>30-50</td>
<td>eastern hill zone front of mountain</td>
<td>mainly buried in the depth depth:20-100(m)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. sand, gravel, clay of alluvial, pluvial &amp; hill side deposit, sand gravel and clay of glaciation sediment alluvial sediment</td>
<td>10-50</td>
<td>down part of the plain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Middle Pleistocene</td>
<td>2. sand, gravel, clay of alluvial, pluvial &amp; hill side deposit, sand gravel and clay of glaciation sediment alluvial sediment</td>
<td>10-40</td>
<td>from the zone in front of the mountain to the down part of plain</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper Pleistocene</td>
<td>1. sand and gravel with clay 2. gravel 3. sand 4. clay 5. loess-like soil 6. clayey soil 7. bedrock 8. Lower Pleistocene 9. Middle Pleistocene 10. Upper Pleistocene 11. Holocene</td>
<td>1. The wave velocity value of foundational medium in the area: From the data of practical surveys in the area, together with the materials of that in the plain down Liao River and Haichen alluvial and fluvial fan, it is advanced that the numerical value of the wave velocity of transverse wave roughly corresponds to that of the soil in the site (Table 4). There is the similarity between the lithofacies of the site in Fushun City and that in Shenyang City. The structure of soil is loose, the wave velocity is quite low, and the capability of withstanding earthquake shock is not so good.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Characteristics of Lithofacies of the site and velocity of Transverse Wave in Shenyang Area.
2. The characteristics of the value of shearing modulus (Go) of the ground in the site: The value of the average shearing modulus (Go) in the site is a comprehensive representation of rigidity, thickness, filtration and amplification of the soil bed. On the basis of lithofacies and according to value Go, the area will be divided into four regions and ten districts in this paper. In order to distinguish the difference in the influence of ground characteristics on the seismic damage, a trend plan of the change in value of Go in the city district is drawn based on the data on Quaternary system in the Shenyang City district (Fig. 6).

From the plan it can be seen that the value Go is decreasing gradually from east to west in Shenyang City, i.e. from \(24 \times 10^3\ t/m^3\) in the east of \(21 \times 10^3\ t/m^3\) in the west. Value Go of the alluvial district of the present valley in the southern part of the City district decreases apparently. The wave velocity of transverse wave and the mass density of soil beds referred in this paper can be seen in Table 5.

C. The main problems of seismic engineering geology in Shenyang-Fushun area and its evaluation.

Through all the analyses about problems of seismic engineering geology which may happen are to be discussed as follows:

(A) The main problems of seismic engineering geology

A sketch of seismic engineering geology in the area has been drawn, the conditions of it there included and a brief evaluation has been made on the likely districts of seismic damage too (Fig. 8).

1. Seismic effect of the fault structures in the area: The difference of the characteristics of seismic engineering geology is subjected to the characteristics of faults.

(1) Seismic cases at home and abroad show that the earthquake that is less than magnitude 6 results seldom in seismic fault and directly in losing efficacy. According to the corresponding relation of maximum acceleration-scale, and the relation magnitude-scale, the three probable dangerous earthquake existing in the area have the elliptical dangerous scale field of degree VII to VIII. Its long axis is 6 to 8 kilometers. The western outskirt of Shenyang City approximates to the field of degree 7 in Yuhong dangerous earthquake.
<table>
<thead>
<tr>
<th>type of ground &amp; rock</th>
<th>age</th>
<th>velocity of transverse wave $V_S$ (m/Sec)</th>
<th>mass density (ton sec$^2$/m$^4$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>artificial soil</td>
<td>Q4</td>
<td>120</td>
<td>0.17</td>
</tr>
<tr>
<td>mud</td>
<td>Q4</td>
<td>120</td>
<td>0.17</td>
</tr>
<tr>
<td>muddy soil</td>
<td>Q4</td>
<td>150</td>
<td>0.19</td>
</tr>
<tr>
<td>loose-like clay</td>
<td>Q3</td>
<td>300</td>
<td>0.19</td>
</tr>
<tr>
<td>hard clay</td>
<td>Q2</td>
<td>320</td>
<td>0.19</td>
</tr>
<tr>
<td>clay sand</td>
<td>Q4</td>
<td>200</td>
<td>0.18</td>
</tr>
<tr>
<td>silt and fine</td>
<td>Q4</td>
<td>120</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>Q2</td>
<td>250</td>
<td>0.18</td>
</tr>
<tr>
<td>middle-sizedto coarse sand</td>
<td>Q2-3</td>
<td>320</td>
<td>0.19</td>
</tr>
<tr>
<td>gravel &amp; fine sand</td>
<td>Q4</td>
<td>320</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>Q1-2</td>
<td>350</td>
<td>0.20</td>
</tr>
<tr>
<td>weathered rock</td>
<td>Q</td>
<td>400</td>
<td>0.22</td>
</tr>
<tr>
<td>fresh rock</td>
<td>Anz</td>
<td>500</td>
<td>0.23</td>
</tr>
</tbody>
</table>

Table 5. Wave Velocity of Transverse Wave and Mass Density of Each Soil Bed

Fig. 6. Trend of Quaternary Structure and Value Go in Shengyang City.
<table>
<thead>
<tr>
<th>sign</th>
<th>districts of lithofacies</th>
<th>average amplified coefficient value $\gamma$ (T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>southeou &amp; northern base rock districts of Hun River</td>
<td>1.0</td>
</tr>
<tr>
<td>II</td>
<td>alluvial &amp; pluvial ancient far region of Hun River</td>
<td>1.3</td>
</tr>
<tr>
<td>III</td>
<td>alluvial district of Hun River</td>
<td>1.5</td>
</tr>
<tr>
<td>IV</td>
<td>basin district of Pu valley</td>
<td>2.5</td>
</tr>
<tr>
<td>IV$_1$</td>
<td>present sedimentary district of the Hun Valley</td>
<td></td>
</tr>
<tr>
<td>IV$_2$</td>
<td>alluvial plain of Liao River &amp; Hun River</td>
<td>2.0</td>
</tr>
<tr>
<td>IV$_4$</td>
<td>alluvial &amp; pluvial plain of Liao River &amp; Hun River</td>
<td></td>
</tr>
</tbody>
</table>

Table 6. The average amplified coefficient adopted in each district of Lithofacies.

Fig. 7. Contour plan of the maximum acceleration of the ground of the site in Shengyang-Fushun area.
(2) The active faults, except those in the three seismic dangerous regions of Yuhong, North Xinchenzi and Suijatun of magnitude 5 to 6, are hard to cause damaging effect on the earth's surface; even future earthquakes will happen in other districts.

(3) Shenyang-Kaizhuan fault zone and that of Hun River both cut the earth's crust for tens of kilometers. They are of large scale. They exert the damping travelling from west to east, i.e. toward Fushun, on the seismic wave of dangerous earthquake. So their existence decreases the influence of scale on Fushun area. But the sites in the city and countryside on the space between Shenyang-Kaizhuan fault and Damintun fault will directly be subjected to the influence of the three dangerous earthquakes, because they have no seismic decreasing or dampening.

2. The seismic effect of buried 'hidden mountain profile':

The amplifying of seismic wave of dangerous earthquake will be remarkably decreased, and the seismic damage will be weakened with the result that a field of degree I may occur in the area of Scale VII to VIII. Because of a buried 'hidden mountain' connected with the eastern mountain land with a thin overburden though Shenyang City lies near the seismic active region of Neocathaysian system, its depth corresponds to the partial hill of upper mantle and the maximum acceleration of its ground movement is a max > 200 cm/sec².

3. In the alluvial district of Hun River and the microrelief along its banks having a silt and fine sand bed in parts of the districts, the depth to ground water is quite shallow. Partial liquefaction and cracks along the banks may occur in the district of degree VII. The seismic damage will be increased and it will threaten engineering structures along the banks.

4. In the region west to Shenyang there are conditions of the dangerous field of degree VII where liquefaction of sand and soil may occur. The depth of ground water is quite shallow, in a considerable part of district. The bed of silt and fine sand is huge. The liquefaction of sand and soil may occur.

(b) Evaluation on seismic engineering geology of the ground in the two cities of Shenyang and Fushun.

1. The city district is directly controlled by the seismic active zone of the Neocathaysian system. There is seismic danger of more than magnitude 5 to its south, north and west. But the city district is not situated straight on the dangerous position of breaking out an earthquake. From Fig. 8, it can be shown that the parameters of ground movement of the rock in the city district are: $a_{\text{max}} > 100$ cm/sec². $T$ (the cycle of the main vibration phase) $t$ (the continued time of strong earthquake) = 10 to 15 sec. Sometimes $a_{\text{max}}$ of the ground reaches 200 cm/sec², partially 250 cm/sec². That corresponds to degree VII to VIII of basic scale value. The city district of Shenyang has good capability of being anti-seismic, owing to the shallow buried depth of bed rock there, the deep buried depth of ground water and the rock being mainly of sand and gravel. The anti-seismic regulation Tj-78 made by the state, degree VII for the basic scale, can be carried out.

2. The city district of Fushun is low in frequency of the seismic activity in history and weak in intensity. At present there is no dangerous district of an earthquake of over magnitude 5. The city district is chiefly influenced by the possible earthquake of magnitude 5 to 6 which may take place in Suijatun, Yuhong and Xinchenzi near Shenyang to its west. The scale of influence is degree VI to VII. The maximum acceleration of the ground is 100-150 cm/sec² and $t$ (the continued time of strong earthquake) = 7 to 10 sec. The city district of Fushun has special conditions of engineering geology. The open-cast mining and underground excavations in Fushun Coal Mine, Hun River's passing through the middle part of the city district, its lying-on the fault depression zone of Hun River and a large Dakuofang Reservoir in the eastern part there, any of the four places may cause unhealthy physical geological process, from the earthquake, adding seismic damage in the city. It is necessary to do comprehensive analysis for engineering sites there together with geological conditions of a certain construction and take appropriate measures for designing-structures of seismic proof and anti-seismic.

The level of seismic danger is determined in the area, on the basis of seismic geology as the base. This paper has done an analysis on the reaction characteristics of ground movement, and an
1. Predicative epicenter of dangerous earthquake
2. Main fault (active)
3. Isopach of the Quaternary System (M)
4. Boundary line of loose formation and bedrock
5. Blind hidden mountain diathesis
6. Isoline of maximum horizontal acceleration (m/sec²)
7. Damaged region of predicative dangerous earthquake (VII-VIII degree)
8. Probable places for liquefaction of sand

Fig. 8. Sketch of Seismic Engineering Geology in Shengyang-Fushun Area.

evaluation on the stability of constructions in that area. There must be some problems to be further investigated. Any comments will be welcome.

Main References


SURFACE FRACTURES CONNECTED WITH THE SOUTHERN ITALY EARTHQUAKE (NOVEMBER 1980)—DISTRIBUTION AND GEOMORPHOLOGICAL IMPLICATIONS

FRACTURES DE SURFACE RELIEES AU SEISME DE L’ITALIE MERIDIONALE (NOVEMBRE 1980): DISTRIBUTION ET IMPLICATIONS GEOMORPHOLOGIQUES

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ABSTRACT

The earthquake, that struck Southern Italy in November 1980, caused many vast surface effects. The main aim of this research is to analyse the typology and distribution of these effects in order to investigate the factors of their location and the implications they have on the morphological evolution of the area.

The examined area is situated in the northern part of the striken belt and was chosen chiefly for its noteworthy surface effects (fractures and landslides).

The azimuthal distribution of the observed fractures are prevailing in the apenninic direction (N140°–150°) in the South-East, and in the anti-apenninic direction (N50°–70°) in the North-West. These distributions show close consistency with those of the main tectonic dislocations (faults largely with quaternary activity), with the drainage network pattern, the isoseismal lines arrangement and, partly, with the solution of the hypocentral mechanism of the main shock. Large and important landslides showed in connection with the fractures.

The shining effects ("fire lines") associated with gas emission, seen by many natives when the fractures opened, and the sharp increase in helium content in soil gases observed across of them would prove the considerable depth of these fractures or, at least, their possible connection with the deepest crust levels. Moreover, the fractures showed a typical recurrence, since they were observed in the same position and with the same effects during past earthquakes.

Observation of aerial photos, foregoing the earthquake, shows the existence of lineaments parallel to the observed fractures and frequently representing their extension for several kilometers. These observations make it possible to associate the location of fractures with deep structural and, perhaps, geodynamic control.

The recurrent seismic fractures must be considered morphogenetically important: they condition both the setting up and evolution of the drainage network and the landslides activation on slopes.

In any case they represent an essential danger component to be taken into account in territorial planning projects.
ABSTRAIT
Le séisme, qui a touché l'Italie du Sud en novembre 1980, a causé plusieurs et considérables effets de surface. Le but principal de cette recherche a été de analyser leur typologie et leur distribution, de façon à réperer les facteurs responsables de leur localisation et les implications sur l'évolution géomorphologique de la région. Le territoire examiné se place dans la partie septentrionale de la zone intéressée par le séisme et son choix a été fait principalement à la suite de l'importance des effets de surface (fractures et glissements). Les distributions azimuthales des fractures observées montrent deux directions nettement prépondérantes: la première apennine (N 140°-150°), dans la partie SE, et la deuxième antiapennine (N50°-70°) dans celle-là NO. Ces distributions sont strictement congruentes avec ceux des dislocations tectoniques plus importantes (failles le plus souvent actives pendant le Quaternaire), avec le réseau hydrographique, la disposition des isostistes et, partiellement (direction apennine), avec le mécanisme hypocentral de la secousse principale. En correspondance avec les fractures se sont manifestés des larges et importants glissements de terre. Les effets lumineux ("lignes de feu") associés à des émissions de gaz, que beaucoup de gens ont observé lors de l'ouverture des fractures, et la remarquable augmentation dans les gaz du sol du contenu en hélium, mesuré le long de quelques-unes d'elles, démontrent leur raisonnable profondeur ou, au moins, leur probable liaison avec les plus profonds niveaux de l'écorce terrestre. De plus, ces fractures se sont eues régulièrement dans les mêmes positions et avec les mêmes effets lors des séismes du passé. Les observations des photos aériennes, précédentes ce séisme, montrent l'existence de linéaments parallèles aux fractures observées, lesquelles en constituent souvent le prolongement pour plusieurs kilomètres. Ces observations font supposer que la localisation des fractures tire son origine d'un contrôle structural profond et, probablement, même géodinamique. Ces fractures sismiques représentent un important facteur de la morphogenèse de la région: elles conditionnent soit la position et l'évolution du réseau hydrographique, soit l'activation des glissements de terre sur les versants. Elles représentent de toute façon un facteur de risque essentiel qui doit être pris en considération lors de l'exécution des projets d'aménagement du territoire.

Introduction

The earthquake which hit the Campanian and Irpinian regions of Southern Italy in November 1980 (1), produced vast and numerous surface effects, such as ground fractures and landslides. Many researchers are studying the problem of the origin as well as of the consequences of these effects (Bollettinari & Panizza, 1981; Carmignani et alii, 1981; Cinque et alii, 1981; Cotecchia, 1981; Lombardi, 1981; Maggiore, 1981; Cantalamessa et alii, in press; Genevois & Prestininzi, in press).

As part of the special programs promoted by the Italian National Research Council, the present work aims at analysing the typology and distribution of these phenomena and at studying their origin as well as their relation to the geomorphic evolution of the area.

The examined area is situated in the Northern part of the zone hit by the seismic event (Fig.1) and was chosen because of the extremely important surface effects which showed there despite the relatively low seismic intensity (VI-VII Mercalli modified; Prog. Fin. Geodinamica, 1981).

(1) The focal parameters of the main shock are: $T_0 = 18:34:52.8$; UTC, Lat. 40°N, Long. 15°18'E; $m_c = 18$ Km; $M = 6.8$ (Scarpa, 1981).

(2) On the MSK scale, fractures in the ground are graded macroseismically between VIII and XII; however, in this connection, Loubreback (1942, pg. 319) points out that:
large fractures were observed (3).

The distribution of surface effects has been compared to the structural setting of the area and its overall geomorphological characteristics. Finally, helium content in soil gas was investigated in correlation with the fractures, with the purpose of obtaining data on the fractures' development in depth.

**Geotectonic frame**

The studied area is situated East of Benevento and, as stated, coincides more or less with the Northernmost part of the zone hit by the earthquake. The area develops in the NW-SE direction following an alignment crossing the towns of Bisaccia-Ariano Irpino-S.Giorgio La Molara (Fig.2).

The geological formations of the area, consisting in a series of overthrust nappes, show extremely complicated stratigraphic relationships due to the tectonic events which have involved the Southern part of the Apennine between the Cretaceous and Pliocene periods (D’Argenio et alii, 1973).

These formations are the result of the evolution of paleogeographic units characterized by different sedimentation facies. Their present relationships are mostly controlled by tectonics (Scandone, 1972). Also the pelitic-sandy Pliocene sediments are affected by faults, showing that tectonic activity has continued after Pliocene age. The Quaternary tectonic activity, proved by geological and geomorphological evidence, is prevalently of a tensile type (Aprile et alii, 1979a-b) and produces a characteristic morphostructural setting in blocks which are limited by normal faults having essentially apenninic (NW-SE) and anti-apenninic (NE-SW) strike. With this tensile phase a general uplift of the area took place which produced a quick deepening of the subsequent linear erosion:

"it is impossible to determine how strong or how slight a disturbance would be necessary to produce the effects".

(3) To determine the azimuthal distribution, fractures with limited linear deve-

---

**Fig.1 – Examined area location: maximum accelerations (g/10), seismic intensity (VI, VI,.. Mercalli modified) and isoseismal lines trend are showed (Berardi et alii, 1981; Prog. Fin. Geodinamica, 1981).

thus the water courses have dismembered a more ancient morphological surface characterized by low relief energy ("paleosurface") and of which at present only some limbs are left over. This structural setting seems consistent with the type of seismic phenomena occurring in this area.

In fact, on one hand the isoseismal lines traced for the last events show a prevailing trending in the apenninic direction, on the other hand focal mechanism are usually of a tensile type (Ritsema, 1969; Karnik, 1971; Cagnetti et alii, 1978). Specifically, the focal mechanism determined by the main shock of the last event, shows a clear extention development have not been taken into account, nor have those which are directly connected to gravitational phenomena and/or phenomena of differential compaction, and finally fractures which developed entire-lyon construction works (such as roads) have not been considered either.
with an anti-apenninic tensile axes (Scarpa, 1981).

**Surface effects**

As said before, among the most striking surface effects produced by the earthquake, the opening of fractures undoubtedly represents the most significant phenomenon. In fact, the fractures have broken up the ground for stretches as long as various kilometers, mostly with straight lineaments, and appear either separately or in groups made up of parallel or at times conjugated systems.

Many fractures are parallel or coincide with the main fluvial axes. However, there are numerous examples of fractures which cross slopes, ridges and shallow areas, regardless of the geomorphological directions.

These fractures do not show noteworthy relative displacements except in some cases, as for instance near Trevico, where relative displacements have been observed which measured up to a few decimeters (Fig.3).

The distribution of fractures has been studied taking into account the direction and the length which corresponds to the "weight" of fractures in statistical analysis. Figure n.2 shows the position of the fractures observed (4).

In Fig.4 we see the azimuthal distribution of these fractures in the entire area, whereby preferential apenninic and anti-apenninic lineaments are clearly visible and, subordinately, also meridian and antimeridian directions show.

A closer examination of the fractures distribution has induced to subdivide the area in two zones: one situated in the NW, the other in the SE. The respective diagrams of the azimuthal directions (Fig.4 b-c) clearly show distinct and areally differentiated maxima. The N140°-150° direction, virtually coincident with the apenninic one, characterizes the SE part of the examined area. The N60°-70° direction with anti-apenninic direction is the one which prevails in the NW part. In both cases data dispersion seems to be relatively small.

One of the main characteristics of most of the fractures is their recurrence with each of earthquakes which have periodically shaken regions of Campania and Irpinia. Numerous direct eyewitnesses for the more recent seismic events (1912, 1930, 1962) as well as the analysis of the literature on the superficial effects of the 1688, 1805 and 1930 earthquakes (Alfano, 1930; Oddone, 1930; Vari, 1930) all show a definite repetition of the phenomena observed at present.

During the survey, evidence was plentiful indicating that the opening of fractures was accompanied by gas emissions besides flashes and shining effects ("fire
Fig. 4 — a) azimuthal distribution of fractures (A) and of rectified directions of fluvial axes (B) for the whole examined area; b and c) azimuthal distribution of fractures (A) and of rectified directions of fluvial axes, 1th order excluded (B), computed separately for NW and SE areas; d) azimuthal distribution of principal tectonic dislocations computed for NW and SE areas.

The examination of aerial photos taken before the seism shows in correspondence with the observed fractures on the ground or parallel to them, clear cut geomorphological lineaments of a length at times as long as tens of kilometers. Such lineaments possibly represent the "lines" along which the fractures themselves periodically reopen.

Numerous landslides phenomena (Fig. 2) of varying typology and often of considerable size were seen to be associated with fractures. These movements which take particularly on pelitic and pelitic-arenaceous slopes, almost always represent a reactivation or a sudden acceleration of already existing processes. Such event have taken place almost in coincidence with the main shock and their evo-
lution in time has been conditioned by the morphological characteristics of the slopes as well as by the geotechnical ones of the involved rocks and soils (Cantalamessa et alii, in press; Genevois & Prestininzi, in press).

In most cases landslides belong to the "earth-mud-flow" type, also associated with rotational slides in the upper part (Varnes, 1978).

In some cases mass-movements have been observed (Genevois & Prestininzi, in press) which were due to the shifting and slow deformation of prismatic blocks bounded by fracture systems. On cohesive soils the instantaneous shocks produced by the earthquake, have caused quick failure phenomena of the "undrained" type, while on solid rock types the collapse phenomena were prevailing.

Analysis of tectonic structure and drainage network

In analysing the orientation of the observed elements, as has been said, two zones were detected in the examined area having different characteristics. Also the relations between the tectonic structural setting and the drainage system are better defined if the analysis are done separately for the two zones.

The azimuthal distribution of the main tectonic dislocations (5) performed for both areas (Fig. 4d) shows two preferential and clearly separate orientations.

In the SE area there are almost only directions comprised between N 130° and N 150°; i.e. in the apenninic direction, while the other directions are definitely subordinated. In the NW area, the prevailing directions are anti-apenninic, N 40°-N 60°, and apenninic N 120°-130°.

The distribution of the rectified di-

(5) To a large extent the examined faults have been enclosed in the official cartography as well as in some recent studies by researchers of the University of Naples. However, it should be pointed out that, given the complexity and nature of outcrops, these dislocations have partially been traced only hypothetically.

rections consistent with the orientation of the fractures and of the tectonic dislocations even though there is a perceptible dispersion of the orientations (Fig. 4a).

The analysis was continued successively separately (Figg. 4b and 4c) for the two areas neglecting the directions pertaining to the axes of the 1st order (to the 1:100.000 scale). This way preferential directions arise more clearly, dispersion being much less emphasized. In the SE area, a maximum is noted in correspondence with the N140°-150° direction, while in the NW area, the maxima correspond to the N0°-10° and N50°-70°.

It is interesting to note how the elimination of the 1st order axes has caused a relative increase in the N0°-10° strike. Such strike does not show, at least in perceptible form, in the distribution of the fractures, or in the one of the tectonic dislocations.

Helium content measurements

During the last years helium content measurements in soil gases have been utilized also for the prediction of seismic and volcanic events (Doering et alii, 1980; Reimar, 1980) as well as for detecting buried fractures (Denton, 1976; Beltrami et alii, 1980).

In fact, helium is an inert, considerably mobile gas continuously produced in the lithosphere and in the mantle by processes of radioactive decay. Helium tends to escape towards the earth's surface and thence towards the atmosphere: the escape velocities in the two systems balance each other off so that helium content at the ground-atmosphere level, considered constant, is used as a reference standard (5.24 p.p.m.).

In order to verify the existence of an eventual deep "control" on the fractures observed on the surface, a long-interval sampling of the soil gas was taken (one sample each Km²). Indeed in Italian areas of geothermal interest (Larderello, Baccano), this method had confirmed a close relationship between positive helium anomalies and conditions of deep frac-
Fig. 5 - Trend of helium content anomalies along a cross-section normal to the fracture (F) near S. Giorgio la Molara.

Sampling (Bertrami et alii, 1980). Gas samples taken at a depth of about 60 cm have been analyzed with a Leak Detector Mod. 120 SSA modified, with which the Unità Nazionale Geotermica of ENEL is endowed, yielding rather significant results.

In Fig. 5, helium values are listed which were observed in one of the cross-sections of 1100 m length, N160° oriented and normal as concerns the fracture observed, near S. Giorgio La Molara, and expressed in p.p.b.; and which represent the anomalies compared to the reference standard (5.24 p.p.m.).

The measured helium content is anomalous as compared to the atmosphere, with the exception of three points. The largest anomalies contained between 500 and 600 p.p.b., coincide with the track of the fracture itself.

It should be remembered that anomaly values in the range of 50 p.p.b. are already considered as being significant.

The other sections which were carried out fully confirm the development of this graph.

Discussion

The consistency of spatial distribution of fractures with spatial distribution of the main dislocations of the area, and the historical recurrence of fractures in the same positions whenever seismic events occur, prove a close relationship with the existing structural setting. The persistence of fracture lines is equally proven by large lineaments observed on the photo pairs which could represent preferential lines along which fractures open locally.

Furthermore, the overall development of the largest fractures, which is on the whole rectilinear and does not seem to be conditioned by minor structural surface details (Fig. 6), would also indicate a deep control of the fractures. Besides helium gas content anomalies in the soil, specially in correspondence with fracture tarses, would prove the extension of fracturing down into lower depths.

Structural discontinuities superficial or deep as they may be can cause different responses to the seismic shaking which entail deformations of such intensity as to cause the breaking up of the ground (Medvedev, 1965). In short, this would
be tantamount to a passive effect of the structural setting impressing its "image" on the topographic surface in the course of the seismic event.

The analysis of the data in Fig.1 (Berardi et alii, 1981), shows that maximum accelerations (3.3 g/10) have been registered at a certain distance from the epicenter, indicating that seismic energy has been transmitted along a preferential line going through the epicenter and having an apenninic orientation.

Also the sudden decrease of maximum accelerations registered in the examined area (0.23 g/10; 0.48 g/10; 0.85 g/10; 0.5 g/10) seems to confirm the existing relationship between deep structures, transmission of energy and superficial fracturing. Besides, also isoseismal lines show sudden changes in their direction (also in previous earthquakes: Majo, 1930; Serva, 1981) and follow roughly the same orientation as the main fractures.

This "structural" interpretation of fractures does not exclude the possibility of "dynamic" effects in the fractures' genesis due to the regional tectonic stress field acting in the area. This much is suggested by the definite consistency of most fractures with the solution of the hypocentral mechanism of the main shock and by the fact that previously considered tectonic structures themselves show Quaternary activity. In this case superficial effects could be induced by the earthquake according to mechanisms suggested by Evison (1963): superimposing themselves on the tectonic stress field acting on the surface, the stresses produced by the shaking can favour the genesis of fractures according to trends which are consistent with this field.

In short, the shocks would abruptly loosen the existing tensions causing the induction of actual "earthquake induced surface faults" (Liü, 1978), having, in such cases, displacements so limited as to be imperceptible to the naked eye.

In this case the fracture recurrence would prove the persistency of the regional stress field to which also the recent tectonic evolution would be due. At any rate, according to available data, it seems very difficult to establish the relative importance of the "structural" and "dynamic" factors even though it seems feasible to presume that their action may be concomitant.

Besides the two above mentioned factors, also effects induced by the morphologic setting on fracturing should be considered. In fact, on the one hand topographic irregularities can produce "magnifications" of the shock effects (Medvedev, 1975), while, on the other, gravitational stress produces tensile efforts which follow the lines of maximum slope gradient, modulating the stresses produced by the earthquake (King & Vita Finzi, 1981). However, this latter mechanism can explain only some of the observed fractures but not their genesis on the whole.
Indeed, in many cases, fractures do not seem to be conditioned by the morphological setting, inasmuch as they cross slopes diagonally or open upon vast flat areas (remnants of the "paleosurface") where gravitational control is completely negligible. All this seems to be confirmed by the aerial photos observation of vast lineaments which often represent kilometer long extensions of the observed fractures.

Whatever their genetic mechanism, the presence of fractures in the ground and their cyclical activity in coincidence with seismic events which acted probably in various successions during the more recent geological times, must necessarily have important geomorphologic implications. Numerous Authors have noted the close relation between surface fracturing and drainage network pattern in various part of the world. Besides, recent studies tend to emphasize the fact that the organisation of the main surface drainage lines are influenced by active fractures (Melton, 1959; Scheidegger, 1979-1980; Wise et alii, 1979).

The correlation between seismic fractures and main axes of the drainage network seems to clearly confirm the above statements. In this connection it is important to underline that the relative increase in the NS strike (Fig.4c) in the azimuthal distribution of the drainage axes higher than the 1st order, reasonably older and more persistent, could be explained with the effect caused, in the recent past, by fractures which have conditioned the setting of the network. The NS strike does not show in any notable form, either in the distribution of present fractures or in the one of Quaternary tectonic dislocations.

Besides conditioning the evolution and setting up of the drainage network, fractures also play a fundamental role in the evolution of slopes by controlling the genesis and reactivation of landslide phenomena, which are often of large proportions and in some cases cause the remnant edges of the "paleosurface" to retreat.

Fractures represent furthermore various risk elements due either to their opening up and producing damages to the above or nearby constructions (Fig.7), or to the movements of masses connected with them. From this point of view it is obvious that detecting these forms is fundamental for any seismic zoning project not only in the examined area, but also in other regions which could be characterized by the same phenomena.

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SEISMOCENIC PHENOMENA IN THE BAIKAL RIFT ZONE

PHENOMENES SEISMOGENIQUE DE LA FENTE DE BAIKAL

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ABSTRACT

Due to an intensive exploration the Baikal rift is known at present as a highly seismic area of the Mongolia-Baikal seismic zone, which is characterized by various gradients of neotectonic movements of crustal surface. The tectonic activity of the region is responsible for a development of geodynamic processes (endokinetic and exokinetic collapses, taluses, landslides, soilflows, etc.) and favours the formation of potential reserves of solid phase of mudflows. The stability of mountain masses at different tremors depends on the earthquake intensity, morphology and steepness of slopes, position in respect to the trend of seismic wave arrival, dampness and state of weathering of grounds, their looseness or viscosity, frozen or thawed state. On the other hand, frequent earthquakes of small intensity (I°= VI) cause changes in the strained state of rocks, lead to a weakening of internal relations in them, i.e. they create conditions for a development of exogenic geological processes. After the Uoyan earthquakes (1976-1977) it was possible to establish in natural conditions an approximate standard of slope stability depending on different seismic actions (M = 5.2 and 4.7; K = 13-14 and 13) and the state of constituting grounds, including an amount of fine-grained filler and exposition. New manifestations of exogenic geological processes which are confined to tectonic active zones evidence a high seismicity of the region.
ABSTRAIT

A l'heure actuelle, la zone d'une grande mise en valeur du rift de Baïkal est un terrain fortement sismique de la ceinture sismique Mongolo-Baïkaliennne, qu se caractérise pas de différents gradients des mouvements néotectoniques de la surface du sol. L'activité-tectonique de la région détermine l'évolution des processus géodynamiques (écoulements, éboulements, glissements du terrain etc...) et contribue à la formation de la morgie éventuelle de phase solide des écoulements de sol. La stabilité des massifs montagneux à des différents secouements dépend de la force du séisme, de la morphologie et de la pente de base des versants, de la position par rapport à la direction d'une onde sismique, l'humidité et l'altération des sols, leur caractère mouvant ou fixe, l'état gelé ou dégelé.

D'autre part, les tremblements de terre fréquemment répétés d'une petite force (à VI balles) permettent de changer l'état tendu des roches, de diminuer les liaisons intérieures et c'est à dire forment les conditions pour le développement des processus géologiques exogènes. Sur l'exemple des tremblements de terre à Ouajan (1976-1977) dans les conditions naturelles e été établi l'étalon approximatif de une caractéristique de la stabilité des versants en fonction de la différente activité sismique (M = 5,2 u 4,7; K = 13-14 u 13), de l'état des sols ou composants y compris la quantité de la charge finement dispersée et l'exposition. La liaison de nouvelles traces des processus géologiques exogènes avec les zones tectoniques actives témoquent la sismicité tre élevé de la région.

The Baikal rift zone is at present intensely explored. The Baikal-Amur railway track, as well as complex industrial enterprises, timber and mining industries, towns, settlements are now being constructed in this region. They need a rational location with regard to all dangerous endogenic and exogenic processes and phenomena.

The geologo-geomorphological structure, high tectonic mobility of the crust show very complex natural conditions in the region. The morphostructural analysis of the present relief indicates that ancient folded structures underwent significant changes resulting from tectogenesis which initiated in the second half of Neogene and was active throughout the Quaternary period. In the present epoch the Baikal rift zone is still active (Florensov, 1960).

The intensive block movements are observed here. The system of rejuvenated faults developed in the upper and middle flow of Khany, Chara, Sjulban, Muyakan, Angarakan, Goudzhekit and Kunerma Rivers is
well-defined in the relief as great and extensive morphological scarps, chains of linear saddlers on water-divides, asymmetrical trenches half-covered by talus, narrow zones of jointing in crystalline rocks.

A complex tectonic development of the Baikal rift was accompanied by origination of thick zones of crushing and formation of roughly-schistose rocks: tectonites, mylonites, cataclasites and breccias. When collapsing they produce waste piles of rock debris: fine-clumpy material easily transported by mudflows. Because of an intensive tectonic crushing advantageous conditions have been created for strong weathering of bedrock. As a result, on mountain slopes there are extensive accumulations of loose-detrital material of different fractions - from fine-argillaceous and sandy particles to fragments and blocks of great dimensions. Tectonic joints and fractures of different character cause an origination of gravitational processes: collapses, taluses, soilflows.

A strengthening of movements occurring in different directions in the present epoch is an important feature of tectonic development of the territory observed. In this case should be mentioned a subsequent increase of velocity of intermontane depression subsiding and uplift of ridges surrounding them. They lead to an increase of the relief energy and intensity of exodynamic processes. From data of V.V. Nikolaev and S.D. Khilko (1971) in the Chara depression the gradient of neotectonic movements equals to $1.4 \times 10^{-8}\text{yr}^{-1}$ and in the Priolekma part of the Tchitkanda-Khanisky fault the gradient of velocity of vertical movements amounts to $1.6 \times 10^{-3}\text{yr}^{-1}$.

In the past the tectonic development of the Baikal rift zone was responsible for a wide manifestation of exodynamic processes. The confinement of processes to active geostuctures is apparent and is due to movements of neotectonic character in regional zones of faults. Since the epicenters of strong earthquakes are concentrated in this area we may regard it as the most seismactive region of Mongolia-Baikal seismic belt (Li-ving tectonics ..., 1966). The data on earthquakes which took place during twenty years prove its high seismic potential: the Muya earthquake of November 4, 1957, $M = 7.9$, intensity - X; the Njukzha earthquake of January 5, 1958, $M = 6.5$, intensity - IX; the Olekma earthquakes of September 14, 1958, $M = 6.5$, $I^o = IX$; the Mujakan earthquake of November 11, 1962, $M = 6.5$, $I^o = VIII$; Tasse-Yuryakhsky earthquakes of January 5, 1967, $M = 7.0$, $I^o = IX$; the Uoyan earthquake of November 2, 1976, $M = 5.2$, $I^o = VI-VII$. Above 1000 shocks of different intensity are recorded in this area. The earthquake recurrence on the segment from the Olekma River to Lake Baikal for 1000 yrs is: $I^o = VIII - 250$ times, $I^o = IX - 100$, $I^o = X - 30$, $I^o = XI -
This high seismicity results in a number of geodynamic processes. It influences formation of potential reserves of solid phase of mudflows on account of endokinetic (seismogenic) collapses, displacements of hillside waste, landslides and soilflows.

A number of works (Florensov, 1960; Solonenko, 1963,1980) indicate that moving of slope formations in the process of their moistening by rainy and thawed waters may also begin at comparatively weak earthquakes with intensity IV-V. When speaking about a high seismicity of the Baikal rift zone as a whole we should distinguish segments with seismic activity. These are interdepression mountain links-sites for tunnel constructions. Moreover, the regions of links turned out to be the joints of dislocations with a break in continuity of different directions. They created a complicated block structure of the Earth's crust.

A constant frequent shaking of individual segments of links causes unexpected phenomena. Thus, in seismic observations of 1967 in the region of E.Leprindo Lake (Chara-Muja link) eighty-five earthquakes with intensity VIII were recorded. A steady shaking of glacial deposits leads to a sudden development of thermokarst. The thermokarst ravine 650 m long, 10-15 m wide and 6 m deep was formed. Simultaneously with subsidence on ravine edges solifluctional ground flows took place.

The rift depressions (Verkhnaya Angara, Muya, Chara) are characterized by a low seismic potential. Plastic deformations of the crust are of particular importance in their formation. The joint zones of depressions with mountain rimming possess maximum seismic activity. About 370 earthquakes with intensity VII occurred in 1967 on the southern slope of the Verkhnaya Angara depression. Taking into account a high frequency of small earthquakes, permafrost and a large amount of atmosphere precipitating the landslides, collapses, scree and subsiding of loose slope deposits often occur in epicentral zones. The pronounced collapses of solifluctional terraces of different age, accumulations of hillside waste and beds rocks are well-observed in many places of the Verkhnaya Angara, Northern-Muya and Southern-Muja, Kodarsky and Udokan ridges. Simultaneous slope deformations defined through a dendrochronological method, evidences their seismogenic origin. Besides, catastrophic earthquakes are accompanied by significant distortions of mountain slopes, on which great seismotectonic fissures, cracks of subsidence, etc. appear. The strongest residual deformations of the crust are represented by extensive seismotectonic fissures with vertical displacements and strike-slip faults (Solonenko, 1980). As an example we may con-
sider some results of influence of great earthquakes on the formation of geodynamic processes. On the 27th of June, 1957 there was an earthquake (I°= X-XI, M = 7.9, focal depth = 22 km) with epicentral zone situated in a small Namarakit intermontane depression of the embryonal type, near the south-western margin of the Udokan range. The earthquake was felt over the area of 1 mln. sq.km. The underground shocks with intensity VII were recorded at the distance over 500 km (Chita city). Crustal motions in the same structural field the compression and strain fissures, faults and upthrow faults, shifts of different directions, twisting folds and vortex structures immediately formed. Seismogravitational movements of rock masses occurred on the area of 150 000 sq.km (Living tectonics... 1966).

In 1978, when examining the Namarakit depression and slopes it was noted that the nature in the past twenty years managed "to cure the injuries" produced by the Muya earthquake. It was also found that at this earthquake the Namarakit depression subsided by 5-6 m. As a consequence, on the depression bottom, where a small river was running, a chain of lakes originated. The trees growing on shores were flooded or half-flooded at caving. The grass vegetation covers shores of lakes and mountainous feet. Only in places of still observed dislocations with a break in continuity which delineate the depression there is a thick bushy wood verdure. Favourable conditions were created for trees growing in fissures. These conditions were provided by a change of regime of underground waters, alteration of the heat balance of grounds; distortion of permafrost, etc.

At the period of 20 years the flanges of fissures in loose deposits obtained gentle contours. Overgrowing started on the space liberated by stone avalanches, fault-collapses and other endokinetic deformations and only collapse masses on a dark-grey background of stone-stream accumulations are marked by light spots. The degree of "injury" healing, as well as the state of damage depend on the type of seismic deformations.

On the mountain segment of the epicentral zone there is an intensive development of erosional excavations which are seen in seismotectonic distortions (big fissures, trenches, etc.) and oriented perpendicularly to water-divides. Mudflows are generally formed in these excavations when rainfalls take place. On the one hand, newly-formed drainage system hollows concentrate the slope flow. On the other hand, a great amount of loose-detrital material is constantly supplied from the fault flanges. It should be underlined here, that small fissures oriented across the slope are half-covered.
and are traced only due to a difference and grading of loose-detrital material.

At the Tass-Yuryakh earthquake (January, 1967) with the epicentral part situated in the basin of the Srednyaa Olekma River, because of strong vertical stroke the layer of large blocks was "shaken up". V.P. Solonenko indicates that monoliths, 40 m$^3$ in amount, were turned upside down. The ice regime on rivers was disturbed, and collapses and avalanches took place on slopes. The disturbance of stable equilibrium of loose-detrital material on slopes lead to a formation of sciflows in a spring time and mud-stone streams not particular to this territory. Thus, in the near-mouth part of the Tass-Yuryakh River on the slope 25-30° steep there was a ground flow 30 m wide and 150 m long. Seismogravitational processes and phenomena are rather different and interconnected. By force and destructive capacity they may cause disaster or prepare conditions for formation of catastrophic exodynamic or fluvial processes. Thus, a failure in the Angarakan River head partitioned off the river bed and presented a potential possibility for a mudflow to form. If the link is disturbed the Angarakan River valley will be a chute directing the mudflow, which destructs everything within its limits. Data on such phenomena are available in literature (Flafker, 1971; Vidal, 1976, etc.).

The high seismic activity of the Baikal rift was noted in the past as well, that is evidenced by the occurrence of collapsed masses of endokinetic origin, which are distinguished by a great amount of collapsed masses and, as a rule, are confined to zones of large tectonic disturbances (Trzinsky, 1979).

It is apparent that the stability of loose-detrital material or weathered bedrock on slopes at a various degree of tremor depends on the earthquake intensity (the main indication) and some other values: morphology, steepness, exposition, dismembering of slopes, their position in respect to direction of seismic wave arrival, dampness, season of year (thawed or frozen grounds) and, at last, degree of weathering and thickness of loose-detrital material, looseness, viscosity, etc. (Iaperdin, Trzhtsinsky, 1977).

The first approximate standard for slope stability at seismic action was obtained in natural conditions from results of epicentral zones of the Uoyan earthquakes, which occurred in November 2, 1976 and in June 4, 1977. Their parameters are: M = 5.2 and 4.7 and K = 13-14 and 13. The difference in magnitude was only 0.5 units, but in the first case it was big enough for small subsidences to form on slopes, though at the earthquake of November the loose-detrital material was locked by permafrost. The most favourable
conditions for displacement of material were observed at the earthquake of June, 1977. In this period ground on southern slopes thawed to the depth of 0.5-0.6 m and on taluses to 1 m. At this time slopes were essentially damp owing to thawing. Thus, in June, 1977 there was the best environment for material to displace from slopes, but visible shifts of the material were not observed, since the force of the shock was, likely, not sufficient enough.

The examination of sequences of the Uoyan earthquakes was performed in three stages. Particular attention was paid to the epicentral zone of the first earthquake, because significant displacements of material had been noted on slopes. The exploration conducted on the 10-th of November, i.e. immediately after the earthquake showed that in the epicentral part on slopes covered with ice there were dark zones which appeared as a result of crumbling and falling of stones from exposed scarpers. Kochetkov V.M. and Zhilkin K.M. reported that individual fragments, 20-30 kg in weight, covered the distance of 60-70 m and reached the mouth of the Anamakit River. Stonefalls formed at different heights from the valley bottom. The area of their occurrences varied from 5 to 35 m and the distance depended on a steepness of slope and coefficient of its surface roughness. The most advantageous surfaces for sliding were the cones of screes locked with ice. Its presence in bodies of taluses accounts for the fact that thick screes situated on slopes with the angle 25-30° showed no reaction to shocks. The basic mass of collapse was due to falling of stones from outcrops. The earthquake caused snow avalanches, above 500,000 cub.m in amount, which were not peculiar to this period of a year.

The repeated exploration of the Anamakit River valley was carried out in February, 1977. Aerial photographic survey and aerovisual works evidenced that slope processes discovered at the first survey (screes, stone-falls, small failures) were of seismogenic character, because their traces were not found at a repeated examination, that is during winter time the slopes were, in fact, stable in natural conditions.

In summer of 1977 the detailed study of processes caused by earthquakes was conducted in the epicentral zone. It should be mentioned that on edges of the Anamakit River valley ancient masses of different age were aerovisually explored. They are seen in the relief as terraces, sometimes thrown back towards slopes or are separated by trenches. These trenches dissect friable deposits. The "seismic relief" was formed at a short time from the geological viewpoint, i.e. after the last glaciation. Due to difference in age of collapse masses we may con-
clude on frequent recurrence of strong earthquakes in this area, for the amount of seismogenic collapses significantly exceeds that of failed masses after the Uyuan earthquake with intensity 6-7.

At the Uyuan earthquake of November 2, 1976 the morphologically similar slopes variously reacted to shaking. This depended on the geological structure, character and properties of loose-detrital accumulations. On the other hand, stability of loose-detrital material is essentially dependent on the ability to retain moisture. Strongly damped crumbled accumulations turned out to be completely frozen and represented an entire monolith. But at the same time individual fragments and blocks of rocks were noted to fall and slide from original outcrops. At flooding of June 17, 1977 on the Anamakit River the lower parts of taluses were undermined. As a result the overhanging caps formed, which were composed of monolithic frozen crumbled accumulations. The destruction of caps occurred through collapsing of great blocks. During the earthquake on neighbouring sites with the same gradients of surfaces but composed of large-block material, significant displacements of the upper layer took place. An increased reaction to the seismic shock in this case is accounted for the absence in the near-surface layer of talus of fine-grained material, which is an important cementing component and coherent link in ground freezing. In the first case a comparative stability of the slope is associated with a large quantity of ice in the talus body. The ice constituted the third part of volume of the crumbled body in the process of undermining. Over 45000 cb. m of material might have displaced to the Anamakit River at the seismic action on the thawed scree. As a consequence of frequent earthquakes, small ones included (which occurred during 20-30 yrs), the mountain slope became "alive". In addition to collapse-crumbled cones there are conglomerations of material removed by ground avalanches.

The above examples show that the most active gravitational processes gravitate towards tectonically active zones. The earthquake epicenters are confined to these zones as well. Thus, exogenic geological processes are regarded as peculiar indicators of activity of seismotectonic zones.

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SEISMOTECTONICS AND SEISMIC RISK STUDY IN AND AROUND DELHI REGION

SISMOTECTONIQUE ET ETUDE DE RISQUE SISMIQUE EN ET AUTOUR DE LA REGION DE DELHI

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ABSTRACT

In this paper seismotectonics and seismic risk of Delhi region has been studied which forms one of the interesting seismic zone of the Indian Subcontinent. This region is located near the trijunction of the Lahore-Delhi ridge, Aravalli-Delhi axis of folding and Delhi-Haridwar ridge. Surrounding Delhi, the epicenters are aligned in N-S direction probably related with Sonepat-Sohna fault.

Focal mechanism solutions suggest that the earthquakes in this region are caused by normal faulting and as such the area is under tension due to wedging of Delhi axis of folding into the lesser Himaleyas (Srivastava 1981).

Landsat image lineaments map of the region clearly reflects the trends of major geological features and several other tear faults.

The slope of the recurrence curve (b-value) has been calculated as 0.35 ± 0.04 and return periods for the earthquake of magnitude 7.0, 6.0 and 5.0 are found to be 65 years, 14 years and 6 years respectively. Coefficients of this recurrence curve have been used in calculating seismic risk in terms of probability in percentage equal to a certain magnitude for design periods of $T = 1$ year, 10 years and 50 years using a method as described by Bath (1979). Significant design period of fifty years shows 100% probability of occurrence for magnitude 6.0 and 80% probability is shown by magnitude 7.0. Therefore, the risk equivalent to magnitude 7.0 is to be taken on safer side in all civil construction in this region for which acceleration value is 0.15 g.
ABSTRAIT

Dans cet article Sismotectoniques et risque sismique de la région de Delhi on a étudié ce qui forme une des zones sismiques intéressantes du subcontinent indien. Cette région est située près de la trijexion de l'arête, de Lahore-Delhi, l'axe de pliage Aravalli-Delhi et l'arête de Delhi-Hardwar. Entourant Delhi, les épicentres sont alignés à la direction Nord-Sud probablement ayant rapport à la faille Sonepat-Sohna.

Des solutions mécanismes focales suggèrent que les tremblements de terre dans cette région sont causés par la faille normale et comme tel la région est sous tension due à coinçage aux Himalayas secondaires (Srivastava 1981).

La carte linéairement de l'image Landsat de la région clairement reflète la tendance des traits majeurs géologiques et beaucoup d'autres abîme des failles. La pente de la courbe périodique à été calculée comme 0.35 + 0.04 et les périodes de retour pour les tremblements de terre des grandeurs 7.0, 6.0 et 5.0 sont trouvées d'être 65 ans, 14 ans et 6 ans respectivement. Des coefficients de cette coube périodique ont été employés pour calculer le risque sismique sous forme de probabilité en pourcentage égale à une grandeur certaine pour des périodes de desse in τ = 1 I an, 10 ans et 50 ans employant une méthode décrite par Bath (1979). Période de desse in significante jusque 50 ans montre 100% occurrence de probabilité pour une grandeur 6.0 et 80% de probabilité, est montrée par une grandeur 7.0. Par consequent, on doit prendre le risque équivalent à grandeur 7.0 à un côté plus sur pour toutes les constructions civiles dans cette région pour lesquelles la valeur d'accélération est 0.15 g.

INTRODUCTION

Region in and around Delhi forms one of the interesting seismic zone which is transversely aligned with respect to the main Himalayan Seismic zone. Available seismic history of the region as far back 893 AD (Maximum magnitude recorded equal to 7) reveals high seismicity and tectonic instability of the region. Most of the shocks are interpreted to be shallow focus and have locations to the west of Delhi. Maximum concentrations are around Sonepat, Rohtak and Gurgaon. The main cause of seismicity is attributed to the numbers of fractures and faults resulting due to wedging of Aravalli-Delhi axis into the lesser Himalayas, Valdiya (1976). In particular trijunction of Delhi-Hariwar ridge, Lahore-Delhi ridge and the Aravalli-Delhi axis of folding is the most seismically active zone as also observed by Chouhan (1975). Important earthquakes that occurred in and around Delhi are shown in Table I.

Table I shows the active prolonged seismic history of the region with frequent occurrence of
TABLE - I

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Date</th>
<th>Location</th>
<th>M.M. Intensity/Magnitude (M)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>893 or 894 ?</td>
<td>Not far from Delhi</td>
<td>Severe, Many people died (XI or XII)</td>
</tr>
<tr>
<td>2</td>
<td>1720 July 15</td>
<td>Near Delhi</td>
<td>XI</td>
</tr>
<tr>
<td>3</td>
<td>1764 June 4</td>
<td>Bank of Ganges</td>
<td>IX</td>
</tr>
<tr>
<td>4</td>
<td>1825 March 22</td>
<td>Near Delhi</td>
<td>Sharply felt, Rumbling sound heard (VII)</td>
</tr>
<tr>
<td>5</td>
<td>1830 July 17</td>
<td>Near Delhi</td>
<td>Sharply felt, Undulation seen on ground (VIII)</td>
</tr>
<tr>
<td>6</td>
<td>1831 Oct. 24</td>
<td>Near Delhi</td>
<td>Mildly felt, trees were seen swaying (V)</td>
</tr>
<tr>
<td>7</td>
<td>1842 July 4</td>
<td>Near Delhi</td>
<td>Violent trembling felt (VI)</td>
</tr>
<tr>
<td>8</td>
<td>1852 March 31</td>
<td>Near Meerut</td>
<td>-- ?</td>
</tr>
<tr>
<td>9</td>
<td>1911 Oct. 14</td>
<td>31°N-80.5°E</td>
<td>6.7</td>
</tr>
<tr>
<td>10</td>
<td>1934 April 14</td>
<td>20°N-75°E</td>
<td>5.0</td>
</tr>
<tr>
<td>11</td>
<td>1935 March 6</td>
<td>29.45°N-80.15°E</td>
<td>6.0</td>
</tr>
<tr>
<td>12</td>
<td>1937 Oct. 20</td>
<td>31°N-78°E</td>
<td>6.0</td>
</tr>
<tr>
<td>13</td>
<td>1956 Oct. 10</td>
<td>28.15°N-77.67°E</td>
<td>6.7</td>
</tr>
<tr>
<td>14</td>
<td>1958 Dec. 28</td>
<td>30°N-80°E</td>
<td>6.3</td>
</tr>
<tr>
<td>15</td>
<td>1960 Aug. 27</td>
<td>28.2°N-76.7°E</td>
<td>6.0</td>
</tr>
<tr>
<td>16</td>
<td>1964 Oct. 3</td>
<td>Near Sonipat</td>
<td>4.7</td>
</tr>
<tr>
<td>17</td>
<td>1966 June 20</td>
<td>28.7°N-76.6°E</td>
<td>4.7</td>
</tr>
<tr>
<td>18</td>
<td>1966 Aug. 15</td>
<td>28.47°N-78.93°E</td>
<td>5.6</td>
</tr>
<tr>
<td>19</td>
<td>1971 Jan. 27</td>
<td>Near Rohtak</td>
<td>5.3</td>
</tr>
<tr>
<td>20</td>
<td>1975 Nov. 6</td>
<td>29.8°N-78.3°E</td>
<td>4.8</td>
</tr>
<tr>
<td>21</td>
<td>1978 Oct. 16</td>
<td>25 km SW of Delhi</td>
<td>3.4</td>
</tr>
<tr>
<td>22</td>
<td>1979 April 19</td>
<td>70 km North of Delhi</td>
<td>3.5</td>
</tr>
<tr>
<td>23</td>
<td>1980 March 13</td>
<td>65 km North of Delhi</td>
<td>3.0</td>
</tr>
</tbody>
</table>

Disastrous earthquakes of magnitude above 6. Therefore, it becomes necessary to study the seismotectonics and seismic risk involved in the region. The objectives of this paper are to study the seismotectonics of the region using techniques of focal mechanism analysis, landsat image analysis and probabilistic approach of earthquake occurrence for seismic risk evaluation. For the risk analysis, reliable data from 1956 to 1980 have been collected from various sources and the area covered is from 28°N-30°N to 76°E-78°E around Delhi as shown by thick line in Fig. 1.

**TECTONIC SETTING OF THE REGION**

The area surrounding Delhi is
mostly covered by alluvium. Outcrop of Alwar quartzites are observed in and near Delhi and the area lying southwards. The quartzites are in general highly jointed and folded. According to Negi and Ermenko (1968), the following tectonic units can be recognised in the region (Fig. 1): (i) Frontal folded zone, (ii) Areas of Delhi folding, (iii) Rajasthan shelf, (iv) Lehore-Delhi ridge, (v) Punjab shelf, (vi) Delhi-Haridwar ridge, (vii) West Uttar Pradesh shelf, and (viii) Sarda depression.

The Delhi-Haridwar Ridge is seemed to be prolongation of the NNE-SSW directed Aravalli Mountain, as horst delimited by faults. Besides, Hukku (1966) postulated several radial tear faults which may cut through the Aravalli basement and the overlying Vindhyan formations. One such fault is Sonepat-Sohna fault which tapped a hot water spring at Sohna. The Moradabad fault fault delimiting the western margin of
the Sharda Depression is thought to be the extension of the great Boundary fault of eastern Rajasthan which separates the mildly disturbed Vindhyan from the complexly repeated folded Aravalli group.

Epicentre distribution map (Fig. 1) supports a N-S aligned seismic zone possibly related with Sonipat-Sohna fault. Besides, activity is also concentrated along Moradabad fault and along axis of Delhi folding which extends into the same direction beyond Delhi in a linear fashion (NE-SW direction) and into the lesser Himalayas. Chouhan (1975) has suggested that the most active area of this region is the trijunction of the Delhi-Haridwar ridge, Lahore-Delhi ridge and the axis of Delhi folding. Focal mechanism solutions (shown in Fig. 1) of two events marked by asterisk in Table I, obtained from P-wave first motion data from WWSSN seismic records show normal faulting with slip vector direction as NE-SW. Such tensional nature of focal mechanism solutions can be very well explained by tight wedging of the blocks along dislocations converging towards Delhi, Srivastava and Somayajulu (1966).

LINEAMENTS STUDY FROM LANDSAT IMAGERY:

Fig. 2 shows photolinear map of the region in and around Delhi covering in six frames of Landsat. This map has been prepared using diapositive of photo images of bands 5, 7 (black and white) and False Colour Composite in 1:1 m scale.

Photolinears represent aligned topographic features and other geomorphic features, which are indicative of fracture, joint or fault systems. Sharp linear tonal contrasts are also included on the map as linear features if they are identified on two or more sets of images taken at different times of the year. Linear features commonly associated with bedding in sedimentary rocks are excluded unless the bedding is offset or appeared to be truncated along a persistent linear zone. Linear man-made features are identified to avoid misinterpretation. Although the linear features are supposed to define fractures, joints or faults, the geological reasons for most of the linear features observed on the Landsat images are not known. However, in order to explain them, careful ground geological check should be carried out.

Here, the major geological features viz; Lahore-Delhi ridge, Delhi axis of folding, Delhi-Haridwar ridge and the Himalayan frontal folded zone are clearly reflected in the regional trends of photolinears. Criss-cross linears near Delhi shows the complexity of the region probably due to the conjoining of the above mentioned geological features. A lineament running in North-South direction from Sohna to the West of Delhi is the Sohna fault (Valdiya, 1976).

Several criss-cross lineaments are observed in the North-East part of the map near Himalayan folded
zone. These may represent faults and fractures originated due to the tight wedging of the block along displacements converging towards Delhi.

SEISMIC RISK ANALYSIS

For the purpose of calculating seismic risk, recent method of Bath (1979) has been used which is described briefly as below:

Assuming Poisson distribution of earthquake events, C. Lomnitz (1974) has defined seismic risk \( R \) as follows:

\[
R = 1 - \exp(-\frac{T}{\tau})
\]

where \( T \) is the recurrence period and \( \tau \) is the reference period for which \( R \) is to be calculated. This recurrence period \( T \) can be obtained, by regression analysis between number of earthquakes and the magnitude,

\[
\log N = (a - bm) \pm \delta
\]

In eqn. (2), the summation sign in front of frequency \( N \) indicates total

Fig. 2. Landsat Photolinesars Map of Delhi Region
number of earthquakes with magnitude \( \geq M \) for an area of relative size \( 1/S \) and a time interval \( 't' \). Where, \( 'S' \) is the factor for reducing to chosen unit area, \( 't' \) is the observed period in years, and \( 's' \) is the standard deviation for \( \log ZN \).

Further eqn. (2) can be transformed into the following form, referred to unit area and unit time (one year),

\[
\ln(S \Sigma N/t) = (a'-b'M) \pm \delta' \tag{3}
\]

Parameters of eqn. (2) and (3) are related as follows:

\[ a' = (a+\log S-\log t) \tag{4} \]

\[ b' = b/0.4343 \]

\[ \delta' = \delta/0.4343 \]

Considering \( 1/T = S \Sigma N/t \) and putting the reference period \( T=1 \) year, the combination of eqns (1) & (3) yields the following expression of risk \( 'R' \) in terms of \( a' \), \( b' \) and \( M \):

\[
R = 1 - \exp[-\exp(a'-b'M)] \tag{5}
\]

Eqn. (5) gives seismic risk for an earthquake of magnitude \( M \) for a period \( T=1 \) year. Other reference period can be easily introduced by replacing \( a' \) by \( a'+\log T/0.4343 \). Here the calculation of seismic risk is not based on recurrence period for single magnitude but instead observations are combined above some magnitude limit into least square solution. The resulting recurrence period and seismic risk are expected to be more representative due to smoothening involved.

RESULTS AND DISCUSSIONS OF SEISMIC RISK ANALYSIS

The above procedure has been applied to calculate seismic risk in and around Delhi region. \( a, b \) and values have been obtained by regression analysis between \( ZN \) and \( M \). Calculated seismic risk \( R \) for \( T = 1 \) year, 10 years and 50 years have been tabulated in Table II.

Alternatively \( T \) can also be calculated for given values of \( R \) and \( M \) using the following transformation of eqn. (1) and (3)

\[
\ln \frac{1}{R} = \ln[-\ln(1-R)] - a' + b'M \tag{6}
\]

Using the equivalences between (as given by Bath, 1978b) magnitude - intensity and intensity-acceleration for average ground condition, risk can be calculated for certain intensity \( (I_o) \) or for certain acceleration \( (a_o) \). Where, \( M \) is surface wave magnitude, \( I_o \) is epicentral intensity in MM scale, and \( a_o \) is epicentral acceleration in cm/sec^2.

### Equivalences as given by Bath (1978b)

<table>
<thead>
<tr>
<th>( M )</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
<th>3.5</th>
<th>4.0</th>
<th>4.5</th>
<th>5.0</th>
<th>5.5</th>
<th>6.0</th>
<th>6.5</th>
<th>7.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>( I_o )</td>
<td>3.5</td>
<td>4.0</td>
<td>4.5</td>
<td>5.0</td>
<td>5.5</td>
<td>6.0</td>
<td>6.5</td>
<td>7.0</td>
<td>7.5</td>
<td>8.0</td>
<td>8.5</td>
</tr>
<tr>
<td>( a_o )</td>
<td>4</td>
<td>6</td>
<td>9</td>
<td>13</td>
<td>20</td>
<td>30</td>
<td>43</td>
<td>56</td>
<td>77</td>
<td>98</td>
<td>150</td>
</tr>
</tbody>
</table>

VIII.83
Recurrence relation obtained for Delhi Region (Fig. 3):
\[ \log \Sigma N = (2.34 - 0.35 \times M) \pm 0.04 \]

### TABLE II
Seismic Risk Values in terms of percentage probability of occurrence.

<table>
<thead>
<tr>
<th>M</th>
<th>For ( T = 1 ) year</th>
<th>For ( T = 10 ) years</th>
<th>For ( T = 50 ) years</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a' = 2.21, b' = 0.81 )</td>
<td>( \delta' = \pm 0.09 )</td>
<td>( a' = 4.51, b' = 0.81 )</td>
<td>( \delta' = \pm 0.09 )</td>
</tr>
<tr>
<td>2.0</td>
<td>83 ± 3</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>2.5</td>
<td>69 ± 3</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3.0</td>
<td>55 ± 3</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3.5</td>
<td>41 ± 3</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>4.0</td>
<td>30 ± 2</td>
<td>97 ± 1</td>
<td>100</td>
</tr>
<tr>
<td>4.5</td>
<td>21 ± 2</td>
<td>90 ± 3</td>
<td>100</td>
</tr>
<tr>
<td>5.0</td>
<td>15 ± 1</td>
<td>80 ± 3</td>
<td>100</td>
</tr>
<tr>
<td>5.5</td>
<td>10 ± 1</td>
<td>65 ± 3</td>
<td>100</td>
</tr>
<tr>
<td>6.0</td>
<td>7 ± 1</td>
<td>51 ± 3</td>
<td>97 ± 2</td>
</tr>
<tr>
<td>6.5</td>
<td>5 ± 0</td>
<td>38 ± 3</td>
<td>90 ± 3</td>
</tr>
<tr>
<td>7.0</td>
<td>4 ± 0</td>
<td>27 ± 2</td>
<td>79 ± 3</td>
</tr>
<tr>
<td>7.5</td>
<td>3 ± 1</td>
<td>19 ± 1</td>
<td>65 ± 3</td>
</tr>
</tbody>
</table>

![Graph](image)

**Fig. 3.** Recurrence Curve of Delhi Region (Data: 1956-1980).
Fig. 4. Variation of Percentage Probability Risk values versus Magnitude for $\tau = 1$ year, 10 years and 50 years.

Table II (and Fig. 4) shows that there is very less probability for the occurrence of higher magnitude for $\tau = 1$ year. The highest magnitude that can be expected to reoccur with high probability is 2.5. For $\tau = 10$ years the expected highest magnitude is 4.5 and for $\tau = 50$ years the expected highest magnitude is 6.0. However, there is 80% probability that a magnitude of 7.0 can reoccur. Thus a magnitude of 6.0 will reoccur with 100% probability in the next coming 50 years and will be associated with intensity VIII and acceleration value equal to 0.1g. But as there is 80% probability involved with magnitude 7.0, we should consider seismic risk equivalent to this magnitude. This gives probable epicentral M.M. intensity equal to IX and acceleration value equivalent to 0.15 g.

CONCLUSIONS

Region in and around Delhi is seismically very active and the maximum magnitude observed so far is 7.0. Most of the events are located in the trijunctural area of the Lahore-Delhi ridge, Delhi axis of folding and the Delhi-Haridwar ridge and are occurring at shallow depth. Focal mechanism solutions suggest the nature of genesis fault as tensile one. Approximate north-south alignment of seismic epicenters near Delhi-Rohtak-Gurgaon is probably associated with the Sohna fault.
Seismic risk analysis using method of Bath (1979) suggests 100% probability of magnitude 6.0 to reoccur in the next 50 years and 80% probability for magnitude 7.0. Thus to be on safer side seismic risk value equivalent to design magnitude 7.0 is to be taken, which gives epicentral acceleration equivalent 0.15 g.

ACKNOWLEDGEMENT

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EXPERIENCE IN SEISMOTECTONIC INVESTIGATIONS FOR THE EVALUATION OF DESIGN EARTHQUAKE FOR MAJOR ENGINEERING STRUCTURES, W. INDIA

EXPERIENCE EN INVESTIGATIONS SEISMOTECTONIQUES AFIN D’EVALUER LA PROBABILITE DE SEISMES POUR LES CONSTRUCTIONS DES STRUCTURES MAJEURS, INDE

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ABSTRACT

Experience in seismotectonic investigations for the evaluation of design earthquake for major engineering structures has indicated that due to lack of detailed and sufficient geologic, tectonic and seismological data, certain conservative assumptions are made for determining the seismic coefficient for the structure. In view of the safety and economy of the project, realistic value of the seismic coefficient has to be ascertained well in advance by a multidisciplinary approach.

A few examples from the projects located in north-western Himalaya and Peninsular India (Narmada-Tapi Rift Zone) have been cited to stress upon the need for detailed seismotectonic investigations.

ABSTRACT

L’expérience en investigations séismotectoniques afin d’évaluer les probabilités de séismes quand il s’agit de faire le plan d’une structure majeure, a révélé que l’insuffisance en données géologiques, tectoniques et séismologiques conduit à un manque de précision dans la détermination du coefficient séismique de la structure en question. Pour que le projet soit économique et réponde aux normes de sécurité, la valeur réelle du coefficient séismique doit être calculée bien avant de commencer la construction, et cela par une démarche multidisciplinaire.

Nous citons quelques exemple de projets situés dans l’Inde péninsulaire (Zone de faille de la Narmada-Tapi) et dans l’Himalaya afin de souligner le besoin d’investigations séismotectoniques détaillées.
Introduction

During the last three decades there has been many fold increase in the construction of medium and major projects both in the Himalaya and in the Peninsular India. One of the most important inputs during the planning and designing of the project is the knowledge about the geology and seismotectonic history of the area for adopting a suitable design earthquake.

On the basis of the past earthquake history the country has been divided into five seismic zones and this forms a useful guide in adopting seismic coefficients for civil engineering structures (IS 1993; 1975). With the necessity of safe and economic planning of the large dams and nuclear Power Plants, precise geoseismological studies of the area are essential in order to incorporate a suitable seismic factor in the design of the structures.

In the last decade efforts have been made to systematise geoseismological studies in the country as indicated by the case histories of a few selected projects located in the Himalaya and Peninsular India (Fig. 1).

Projects in the Himalaya

A number of dams have been constructed in the terminal gorge of the outer Himalaya. This region is folded, faulted and thrusted
during the Tertiary era and there are evidences that the earth movements are continuing till the Recent. The continental boundaries of the Indian plate are defined by the Kirthar and Sulaiman ranges in the west and north-west, the Himalayan ranges in the north and north-east and Burmese arc in the east (Pal 1972). Himalayan belt is a compression zone with a record of high seismic activity (Fig.2). A number of earthquakes have been recorded and the expected maximum magnitude of the earthquake in the region is as high as 8 on Richter Scale, the maximum intensity is X or more on M.M. Scale and the peak ground acceleration around 50 percent of gravity (Kaila and Rao 1979).

Yamuna Hydel Project

A 55 m high concrete dam across the river Tons and 6.2 km long 9 m diameter tunnel with an underground power house has been constructed under phase I of the Project. Phase II of the Project comprises 5.9 km long tunnel with a surface power house. Slates, quartzites and limestones of Jaunsar Group (Silurian - Devonian age) are folded into syncline and thrust over the Dagshai-Subathu rocks of Eocene to Lower Miocene age (Krol thrust) which in turn are thrust over the Nahan rocks of middle Miocene age (Nahan thrust). Investigations indicated that there have been subsequent movements along the thrusts when the Tertiary rocks were thrust over the sub-Recent deposits. Carbon dating of the material associated with the sub-Recent faults have indicated that these movements might have occurred some 36,000 - 38,000 years ago (Jalote and Jalote 1981).

The second tunnel for the Yamuna Hydel Project would cross the Krol and the Nahan thrusts, active faults and the highly crushed and brecciated intrathrust zone with high mountain pressures. The project area lies in Zone IV of the seismic zoning map of India (IS 1893; 1975) and within iso-seismal VII to VIII on Rossi iso-seismic scale of the Kangra earthquake of 1905 located about 200 km north-west of the project area.
Based on this data, a seismic coefficient of 0.1g has been provided in the design of the structures (Jalote et al. 1975). The second tunnel passing through the squeezing ground and the active faults has been trifurcated with reduction in the diameter of the tunnels, provision of flexible lining and steel supports.

Bhakra dam project

The 225.6 m high straight gravity concrete dam is located in narrow terminal gorge of the river Sutlej to store 1973.56 million cubic metres of water. The Project was commissioned in the year 1963. The foundation rocks comprise steeply dipping Tertiary sandstones and shales which have been intersected by transverse and bedding faults and shear zones. Two regional thrusts lie in the reservoir area. During the Kangra earthquake of 1905, which originated about 30 km north-west of the Bhakra dam site, the area fell within isoseismal VIII on Rossi Forrel scale. The site lies within zone IV of the seismic zoning map of the country and a seismic factor of 0.15g has been taken in the design of the structure (Palta, 1979).

Beas (Pong) dam project

A 132.6 m high earthen dam across the river Beas is located in the Himalayan foothills. The bedrock units at the dam site, folded into minor anticlines, and synclines consist of sandrock and clay shale of the Pinjor formation of the Upper Siwaliks (Pliocene). At places there are evidences that the rocks of the Pinjor formation have been thrust over the Boulder conglomerate of Lower Pleistocene age along the Satlitta thrust. The thrust is located 2.7 km downstream of the dam axis and with an upstream dip of 30° lies 1.5 km below the dam foundation. Three periods of subsequent movements have been noted along this thrust during the Middle Pleistocene, Late Pleistocene and Recent times (Jalote and Tikku 1975) when the Upper Siwaliks were thrust over the Recent to sub-Recent deposits at places.

The area has experienced several earthquakes of magnitude greater than 5 on Richter scale. The Kangra earthquake of 1905 located about 60 km north-east of the dam site was the severest earthquake of magnitude 8 on the Richter scale. The project area fell within isoseismal VII and VIII of the Kangra earthquake. The ground displacements were not recorded along the trace of the thrust. Periodic geodetic survey and high precision levelling is being carried out to know the possibility of present activity along the thrust. Based on the blast test results, 0.12g was adopted as a seismic co-efficient in the design of the earth dam and 0.15g to 0.20g for concrete structures (Jalote and Tikku.
projects in Peninsular India

The Peninsular shield is comparatively a stable region and no mountain building activity has been noticed since the Pre-Cambrian time. Nevertheless the movement of the Indian Plate towards north has resulted in the formation of deep seated lineaments. The Narmada-Son lineament is the most significant fault which has divided the Indian Plate into two main tectonic blocks. These have been further sub-divided into rifts and grabens with central massif (Iyengar, 1977). Investigations indicate that the margins of the Peninsular India have shown sub-Recent movements (Kailasam, 1979). The studies by Kaila and Rao (1979) show that the West Coast is a zone of moderately high seismicity, the East Coast of slightly high seismicity and that Bilaspur-Hyderabad is a zone of low seismic intensity. The Bundelkhand-Ajmer zone has not shown any seismic activity. Koyana earthquake of 6.5 (1979) and a number of other earthquakes have been recorded in the margin of shield area though these are generally of moderate magnitude and frequency. With this background it has now been realised that detailed geoseismological investigations should also be carried out for the projects located in Peninsular India.

Narmada Project

A 138 m high concrete dam is under construction across the Narmada river near Navagam. This is one of the well investigated projects in the country due to its magnitude and close proximity to the Narmada-Son lineament which is considered to be active. Deccan basalts of Cretaceous-Eocene age overlying the infratrappean sediments form the foundations of the dam. A fault along the river channel intersects the dam axis near the right bank. A number of other faults have also been mapped in the area.

The dam site lies in the Narmada-Tapi rift zone trending in E-W to ENE-WSW direction. Neotectonic movements in the area have been recorded recently (Srinivasan et al., 1981). Narmada dam has been considered to lie in a "mobile" belt of about 20 km width bounded by ENE-WSW trending fault towards north and Piplod fault towards south (Project Report, 1981). The maximum magnitude of the earthquake felt in this zone is 6.5 (Narmada earthquake 1846). The Project area falls in the zone III of the seismic zoning map (IS 1893 - 1975). Micro-earthquake studies in the area show evidences of sub-zero and very low magnitude micro-earthquake activity with very shallow depth of foci (upto 5 km) and epicentres randomly distributed around the dam site.

The design horizontal seismic
coefficient as worked out by applying various techniques varied between 0.09g and 0.11g and an average value of 0.1g was recommended. Statistical studies considering the earthquake data of the last 300 years have indicated that during the lifetime of the dam, taken as 100 years, the earthquake which may occur would have maximum magnitude of 5.8 (CWPRS 1979). Recent investigations by the Roorkee School of Research and Training in Earthquake Engineering, Roorkee have recommended an earthquake of magnitude 6.5 assumed to be associated with the Piplod fault located at a distance of about 12 km from the site and depth of focus 18 km (Project Report 1982). However, it has not been possible to establish active status of the Piplod fault. Based on the Koyana earthquake (1967) record, the deterministic approach gives a peak ground acceleration of 0.16g (Project Report 1981). However, it may be mentioned that the Koyana earthquake lies in a different seismotectonic province and it is more than 300 km away from the dam site.

Kakrapar Nuclear Power Plant

An attempt was made to systematically work out the design basis earthquake for the proposed nuclear power plant near Kakrapar in Gujarat. An area within a radius of 300 km around the site was scanned on the Landsat imageries on 1:1,000,000 scale and available large scale aerial photographs to study the regional geological and tectonic features of the area (Mehta 1981). The available data on the past earthquakes was collected to build up a seismotectonic frame-work of the area. On the basis of the studies the area can be divided into six distinct seismotectonic provinces (Fig. 3). The Kakrapar site lies at the triple junction of the Narmada-Tapi, West Coast and Godavari seismotectonic provinces. But the more dominant lineaments in the area are in ENE-WSW direction parallel to Narmada-Tapi lineament. The severest historical earthquake of magnitude 6.5 on Richter scale is located in this province at a distance of about 160 km from the site.

In order to determine peak ground acceleration at the site, with the seismotectonic approach the following factors were borne in mind (IAEA 1979).

1. Where the earthquake of greatest magnitude or intensity has been correlated with a fault or thrust in the same seismo-tectonic province in which the site is located, it is assumed that the epicentre lies on the lineament closest to the site.

2. Where the greatest magnitude earthquake lies in the same seismotectonic province as the site but cannot be correlated with a tectonic structure, it is assumed to lie at the site for the purpose of seismic computations.
Fig. 3: Seismo-tectonic provinces in radius of 300 km around Kakrapar site (Lineaments interpreted from Landsat imageries on 1:1000,000 scale - only major lineaments have been shown).
3. Where the epicentre of the greatest magnitude earthquake cannot be associated with any of the tectonic structures and they do not lie in the same seismotectonic province, the acceleration at the site is determined assuming that the epicentre of the earthquake is at closest point to the site on the boundary of tectonic province.

There are a number of faults sympathetic to the Narmada Tapi and West Coast lineaments but in the absence of sufficient geological and microseismic data, it was not possible to precisely indicate the fault which has been responsible for the highest magnitude earthquake in the area. There are evidences of neotectonic activity in the area to show that the Narmada and Tapi lineaments, bounding the Narmada-Tapi seismotectonic province are active and a few earthquakes might have been associated with crustal adjustment along the Tapi lineament. Thus for computation of the peak ground acceleration, an earthquake of magnitude 6.5 with depth of focus assumed as 30 km and associated with the Tapi fault located at a distance of 30 km from the site has been suggested.

Conclusions

In this paper an attempt has been made to briefly review the seismicity of north-western Himalaya and the Peninsular shield areas with special reference to the design seismic coefficient adopted for the civil engineering structures, based on the geological and seismological studies of these regions.

Himalayan region is a zone of highest seismic activity in India. Several earthquakes have been recorded all along the Himalayan belt extending from Kashmir in the north-west to Assam in the north-eastern part of India. The maximum earthquake magnitude recorded is as 8 on Richter scale. Recent to sub-Recent movements along the major thrusts and faults have also been recorded. Thus the projects located in the Himalaya have greater risk requiring comprehensive geotectonic studies. However, the selection of design seismic coefficient for projects located in the Himalaya are mostly guided by the past seismic history of the area as revealed by a few case histories of the projects cited in the paper.

For a long time the Indian Peninsular shield area was considered to be tectonically and seismically stable. But after the Broach (Gujarat) earthquake of 1970, Koyna (Maharashtra) earthquake of 1967 and the Kothagudem (Andhra Pradesh) earthquake of 1969, it has been realised that the marginal areas of the Indian shield are tectonically active like other shields of the world. There are also evidences of Recent to sub-Recent activity in the Narmada-Tapi, Godavari and Gondwana basins of negative gravity anomaly
with central aseismic zone. The magnitude and frequency of the
earthquakes in these areas are lower than the Himalaya. At the Narmada
and the Kakrapar project sites located in the Narmada-Tapi rift
zone, detailed geoseismological
studies have been carried out to
determine the design earthquake. But still there is scope for more
field studies to understand the
geological and tectonic history
of the area. Further, geological
data is required to be collected
to delineate precisely active
faults, their disposition, length
and order of displacement. It is
also important to determine the
relationship of the earthquake with
faults with greater certainty. In
the absence of this information
the design earthquakes may have to
be based on assumptions.

There are certain constraints
in carrying out systematic geosei-
smological studies particularly in
the inaccessible and difficult
Himalayan terrain. According to
the modern concepts an area in a
radius of 300 km around the site
should be studied in detail for
seismotectonic evaluation. This
is a large area and unless studies
are initiated well in advance by
a multidisciplinary team of geolo-
gists, seismologists and geophys-
sicists, the exact picture may not
emerge during the designing stage
of the project. In recent years
efforts have been made in this
direction but still more studies
are required to be carried out for
each major construction site in
the light of the state-of-art
existing today.

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EVALUATION OF GEOLOGICAL AND SEISMOTECTONIC PARAMETERS FOR A DESIGN SEISMIC COEFFICIENT FOR NAVAGAM DAM, GUJARAT

EVALUATION DE PARAMETRES GEOLOGIQUES ET TECTONIQUES POUR UN PLAN DE COEFFICIENT SEISMIQUE AU BARRAGE DE NAVAGAM, GUJARAT

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ABSTRACT

Geological and seismotectonic studies have been carried out for the 138 m high concrete dam, across the Narmada river, in order to provide a suitable seismic coefficient for the structure.

Deccan basalts (Upper Cretaceous and Eocene) underlain by the infra-trapcean sedimentaries are exposed in the area and have been profusely intruded by basic and acid basalt and dolerite dykes. A 12 m wide fault zone cuts across the dam axis.

The Narmada river lies in a rift zone bounded by faults on either flank with many sympathetic faults and fractures. Geomorphological and Quaternary geological studies have indicated evidences of neotectonic activity in the lower reaches of the river. A number of earthquakes have occurred in this valley in the past. Reference can be made of Broach (1970) earthquake.

Microseismic study around the dam site has revealed 604 minor events in 6 months. Based on the probabilistic approach, a magnitude of 4.2 for a period of 50 - 100 years has been worked out. This study has also indicated active nature of the main Narmada lineament trending in a ENE-WSW direction and located 8 - 10 km away from the dam site.

In view of the complex structural and tectonic setting of the area and its past seismic history, seismic coefficient has to be computed taking into consideration an earthquake of 6.5 magnitude, as the maximum credible one for this dam.
ABSTRAIT

Des études géologiques et sismologiques ont été faites pour un barrage en béton de 138 m de haut sur la rivière Narmada, afin de donner un coefficient sismique convenable à la structure.

Le basalt du Deccan, sous lequel se trouvent les sédiments infrattrappéens de l'âge allant du Crétacé supérieur à l'Eocène, sont exposés dans la région, des basalten acides et basiques d'intrusion y sont abondants ainsi que des veines de dolérite. Une faille de 12 m de large coupe à travers l'axe du barrage.

La rivière Narmada se trouve dans une zone de fissures entourée de failles sur chaque flanc avec plusieurs failles sympatiques et fractures. Des études géomorphologiques et des études de la géologie quaternaire ont mis en évidence une activité néotectonique dans les couches les plus basses de cette rivière. Dans le passé, il y a eu un certain nombre de tremblements de terre dans cette vallée; références Brench (1970) tremblement de terre.

Un relevé microsismique autour du barrage a révélé 604 séismes mineurs en 6 mois. En se basant sur la probabilité, on a calculé une magnitude de 4.2 pour une période de 50 à 100 ans.

Ce relevé a aussi indiqué la nature active de la direction du linéament ENE-WSW de la Narmada placée à 8-10 km du site du barrage.

À cause de la complexité structurale et tectonique de la région et aussi à cause de son histoire sismique, un coefficient sismique est calculé comme maximum crédible pour ce barrage, en tenant compte de séismes de 6,5 magnitude.

Introduction

The 138 m high concrete dam under construction across the Narmada river, near Navagam (21°50' N; 73°45') in Gujarat is a major multipurpose project of India. The dam site is situated within the Narmada-Tapi rift valley - a seismicogenic zone, close to the Narmada-Son linéament and therefore has necessitated a proper appraisal of the tectonic and seismotectonic set up in the area around the dam site, for evaluation of a suitable seismic design.

Geology

In and around the dam site horizontally disposed Deccan basalt flows (Upper Cretaceous to Eocene) varying in thickness from 5 to 55 m and made up of dense as well as amygdaline (Fig.1) varieties, with tuffaceous zones. Besides, thin 'Redoodle' beds are also sandwiched
in between consecutive flows.

Besides, underlying infra-trappean sediments consisting of quartzite, sandstone, shale, chert and limestone are exposed as inliers. The infra-trappeans and basalts are profusely intruded by dolerite and basalt in ENE-WSW direction representing the tectonic impress of the Narmada valley.

These rocks are further dissected by series of joints, shears and faults in ENE-WSW, N-S and NW-SE, directions. Major faults identified in the vicinity of the dam site are Mokhadi, Shekhbar and Surpan, on the upstream side and Akalbar, Sadhu Hut and main Narmada on the downstream side of the dam axis, arranged in an en echelon pattern.

In the river bed, below the dam seat, a 12 m wide zone, trending in N80°E - S80°W with 60° dip towards the right bank is encountered. This fault has vertically displaced the infra-trappean sediments and the overlying basalt, by 210 m, bringing them in juxtaposition. This fault is bound by cross faults viz. Mokhadi on the upstream side and Akalbar on the downstream side.

Tectonic impress

In the area around the dam site forming a part of the lower Narmada valley, a few prominent lineaments identified (Fig.2) are the ENE - WSW - Narmada Son, NNE-SSW - West coast and Cambay graben and NW-SE - Godavari graben. All of these line-
Fig. 2 Lineament map of a part of the lower Narmada valley.
1. Major lineament (300 km), 2. Intermediate lineament (100-300 km),
3. Minor lineament (100 km), showing limits of known faults. 4. Fault.

...ments tend to converge in the area east of Bharuch.

The western part of the Narmada rift valley has been formed due to successive down faulting of the Deccan volcanic blocks, along en echelon faults. The Camay graben was formed during the waning phases of the main Deccan basalt episode. Presence of alkaline basalts east of the Camay graben and of quartz bearing basalt to its west indicate repeated tensional and compressional forces acting in the Narmada - Son lineament.

In the eastern part, the Narmada - Son lineament has separated the Vindhyan (Pre-Cambrian) and Gondwana (Permian-Cretaceous) rocks, to its north and south sides, respectively. The Narmada and Tapi are the only two rivers of the Indian peninsula, flowing westward.

Qureshi (1981) has shown a gravity 'low' along the Narmada rift zone and broad 'high' to its north and south sides, in the gravity anomaly map. Qureshi and Ghosh have attributed these 'low' to the domal upwarp. But, the parallelism in the trends of 'low', 'high' and of structural trends, suggests a broad rift.

The National Geophysical Research Institute, Hyderabad, has carried out Deep Seismic Sounding in N-S direction, (i) from Mehamadabad to Navsari along West coast and (ii)
between Indore and Bhavali. Profile of the earlier survey is available at present and it has shown a broad nature of this rift valley and trace of the main fault along the Narmada river, south of Bharuch. All these faults penetrate down to 'Moho'.

Tectonic activity

Tectonic movements are reported from the Narmada valley, in the area downstream of dam site. Here tilting of terrace blocks away from the river, 10 - 15 m vertical displacement in a terrace block, offsetting of the northerly (Srinivasan et al 1981) flowing tributaries, local convexities in the thalweg, etc., are noticed near the high level Narmada bridge at Bharuch, Tilakwada, Nagrol and Rundh, located on either flank of the Narmada river and suggest tectonic activity in the area.

Seismo-tectonics

In the lower Narmada valley three seismogenic zones are noted with (i) ENE - WSW Narmada-Tapi rift zone, (ii) NNE - SSW West coast lineament and Cambay graben and (iii) NW - SE Godavari graben. Convergence of these lineaments, west of this project area has been highlighted.

Major earthquakes recorded in the Narmada valley are Narmada (1846), Khariwa-Satpura (1938), Anjar (1956) and Broach or Bharuch (1970). The Rewa earthquake (1927) was located close to the Narmada-Son lineament. All these major earthquakes in the Narmada valley were within 100 - 300 km from the dam site. Their magnitudes varied from 5.4 - 6.5 and are attributable to fault breaks along the Narmada-Son lineament. The major Koyna (1967) earthquake occurred 600 km south of the Narmada valley in the Deccan basin terrain.

The Bharuch earthquake was located 100 km west of the dam site. This earthquake originated at the junction of ENE - WSW Narmada-Son lineament and NNE - SSW Cambay graben. The correlation of the depth of focus of this earthquake (Balasundaram et al 1971) with analyzed tectonically active geological features has revealed that this earthquake was probably due to deep seated movements. The iso-acceleration lines of this earthquake (School of Research and Training in Earthquake Engineering, Roorkee, 1971) show distinct pattern of elongation along the ENE - WSW trend of the Narmada-Tapi rift zone, indicating a possible inter-relationship between both of them.

The dam under construction is located in the Narmada-Tapi seismogenic zone aligned in a ENE - WSW direction. Below the dam seat a reverse fault is encountered, with a downthrow towards the left
Fig. 3 Showing geology and locations of epicenters around the Navagam dam site.

1. Alluvium. 2. Basalt flows. 3. Sandstone. 4. Limestone. 5. Dyke
6. Attitude of bed. 7. Fault. 8. Attitude of joint. 9. Micro-
earthquake station. 10. Micro-earthquake test station. 11. Magnitude
0 < O 12. Magnitude O > O

bank. As the dam site is located in the area prone to severe earth-
quakes, on the basis of the estimation of seismic data for the last 300 years
or so and the tectonic setup, the Narmada valley has been included in the
seismic zone III by the Indian Standard Institute with probability
of basic horizontal seismic coefficient 0.04. The design horizontal
seismic coefficient has been worked out to be 0.08 'g'. The Response
Spectra Technique and Amplification Factor and Soil structure Inter-
action methods (Central Water and Power Research Station, Pune, 1969)
have also been used and design horizontal coefficient have been esti-
ated from 0.08 - 0.11 'g'. As the above values are almost similar, the
Standing Committee of the Government of India for advising Design Seismic Coefficient for the River Valley projects adopted (1979) seismic coefficient 0.1 'g', taking geological, seismogenic and tectonic features into consideration.

There have been instances in the world, where large reservoirs like Koyna, Kariba, Kremsta, have triggered seismic activity after impounding. The maximum magnitude of earthquakes generated at Koyna after reservoir impounding is 6.5. But, whether this major tremor could be attributed to impounding is still a mute point. But certain tremors felt earlier have been attributed to reservoir impounding. Therefore, it was recommended that the seismic factor be increased by 25 percent (Central Water and Power Research Station, Pune, 1969) to cover additional risk due to reservoir impounding. Alternatively this dam has been suggested to be designed for an earthquake of 6.5 magnitude.

The School of Research and Training in Earthquake Engineering, Roorkee, has carried out dynamic analysis of the concrete dam and rock fill dyke and has also endorsed a design earthquake of 6.5 magnitude at the nearest seismogenic alignment.

Microseismic study

For monitoring the probable seismic status of the river bed fault, micro-earthquake studies have been carried out by the Central Water and Power Research Station, Pune, in the past and School of Research and Training in Earthquake Engineering, Roorkee, to locate local events associated with faults (Fig.3) and assessment of seismic activity before reservoir impounding. In the first study, five sites were located on hard rock and preliminary analysis of 24 events recorded in 20 days period showed magnitude of earthquakes less than 1.6, within a radius of 10 km from the dam site. Out of these 17 were located on the upstream sides within a stretch of 4 km and the remaining were on the downstream side.

Subsequent microseismic study carried out between February and August 1980 have recorded earth tremors ranging from -1 to 3 magnitude. Out of the total 604 events recorded 172 were above 0 magnitude. Maximum magnitude recorded was less than 3. Because of scattered pattern of events no positive relationship could be established with geological features. Some activity has been recorded along the main Narmada fault located 8-10 km away from the dam site, with 15 km focal depth. Therefore, based on the probabilistic approach an earthquake of 4.2 magnitude has been anticipated within next 50-100 years.

Discussion

The earthquakes in the Peninsular part of India originated due to rejuvenation of
faults. Major uplifts, subsidences (Gomawana rift) and associated faults, fractures, etc. Jai Krishna (1976) has recognised two seismogenetic zones viz. Cambay graben and portion of the Narmada valley from Rajparui fault to Borheedi-Harwani fault. The Rajparui fault trending in a NNE-SSW direction is the nearest fault of the Cambay graben, 50 km west of the dam site. Available geological and seismic data indicate that the river bed fault encountered below the dam seat is a local one, bounded by cross faults on the upstream and downstream sides and may not be capable of generating a major earthquake.

Since the installation of a seismograph at Kevadia in 1973, 35 events 2.5 to 3.5 magnitude have been located within 5 - 15 km focal depths. The major faults viz. Piplod, Surpan and main Narmada, are in the vicinity of the dam site. The main Narmada fault (Narmada-Son lineament) extends eastwards along ENE-WSW direction and is within 8 - 10 km from the dam site. This fault is capable of generating major earthquakes and would govern the ground motion at the dam site, which is situated in 20 km wide mobile belt. Quantitative assessment in this area have shown moderate seismicity in this zone and has also provided good estimate of background seismicity, which may be prospective during impounding period.

The design parameters selected earlier were based on the historical record of the area. Active faults have long intervals (10000 - 50000 years) between shallow focus faulting (Glemons, 1977), associated with damaging earthquakes. The long intervals in combination with short intervals need geological measures, as supplement to historical and seismological records for establishing design events.

The selection of an earthquake of 6.5 magnitude has been made on an idealised 15 km length, assuming vertical and horizontal dimensions approximately same, resulting in an area of 15 sq km of strain build up, responsible for such an earthquake. The 30 km focal depth has been assumed on estimated peak ground acceleration of 0.16 'g' at the dam site. The Central Water Commission has recommended seismic coefficient 0.125 'g'. Dynamic analysis is being carried out.

Acknowledgement

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SEISMIC RESPONSE OF EMBEDDED BUILDING SYSTEMS IN LAYERED SOILS

REPONSE SISMIQUE DES SYSTEMES DE CONSTRUCTION ENFONCES AUX SOLS ASSIS

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ABSTRACT

The design of foundation of structures to resist dynamic loadings due to earthquakes, must meet certain performance criteria. These usually relate to the dynamic response of structures expressed in terms of limiting amplitude of vibration at a particular frequency or limiting value of peak velocity or peak acceleration.

In this paper the dynamic response of structures resting on or embedded in a soil stratum is studied. The various cases analysed is with reference to the dynamic interaction coefficients (horizontal forces and rocking moments) resulting due to the properties of the soil stratum and the motion due to specified ground acceleration resulting due to strong earthquake motion. The study has been made based on the concept of Karasudhi, Keer and Lee (1966) and Parmalee group (1967). The effect of layer thickness and the convergence towards the half space solutions are investigated. The response of the structure with and without embedment is considered to evaluate the response of the structure to soil-structure interaction.

From the results obtained it can be seen that some of the effects studied might be reasonably well reproduced, by applying simple correction factor to the half space solutions and quite comparable with that of the layered system. Also, it is seen from the results that the shear wave velocity, poisson's ratio and embedment effect will have an appreciable influence on the response of the structure resulting due to soil-structure interaction.

*The French version of the abstract appears at the end of the paper.*
REVIEW OF LITERATURE:

The evaluation of the structural response taking into account the soil-structure interaction is a complicated problem. If the structure is assumed to be rigid, (e.g., a nuclear reactor containment vessel) the problem is quite involved from the mathematical point of view and to get the concept of the mathematical idealization of the structure is quite involved. Since the present problem is a wave propagation problem with mixed boundary conditions both displacements and boundary conditions arise. Also there may be rock underlying the soil stratum, which may itself consists of different layers with different properties.

For these reasons, the mathematical model must be a very idealized one. The basic mathematical models so far developed based on the soil-structure interaction is shown in Fig. 1 to Fig. 5. Most of the research work on this topic assumes a perfectly elastic half space, where the only existing boundary for the soil is the free surface.

Generally, these mathematical models have been formulated for the foundations generally adopted to machine foundation vibrations. Reissner (1936), Bycroft (1956), Sung (1953), Kobari et al (1971), have shown the response with respect to a disc or a rectangular element on a half space, subjected to dynamic forces.

This half space theory does not provide a good solution when rock or hard layers are encountered at relatively shallow depths. Warburton (1957) studied the vertical vibration for a rigid circular footing supported by a homogeneous elastic layer that extends to infinity in the horizontal direction and rests on a rigid base. Wass (1972) studied this problem using finite elements and developed the dynamic stiffness matrices for the layered regions of infinite horizontal extent, adjoining the finite elements of irregular region, thereby accounting for dissipation of waves through the layered media.

The influence of the vertical, horizontal and rocking vibration due to seismic excitation with reference to rigid footing or flexible footing proved to be very significant. A rigorous study with reference to the influence of soil-structure interaction is more meaningful in the study of the behaviour of the structure. Rigorous study has been made by Arnold, Bycroft and Warburton (1955), Bycroft (1956) and Warburton (1957) on the horizontal and rocking vibration of circular footings.
wherein all the four modes of vibration are considered. Luco and Westmann (1971) used a suitable method and considered wider range of frequencies. In the formulation, they neglected the coupling between sliding and rocking. The dynamic properties of the soil is determined by Whitman and Richart (1967), Veletsos and Wei (1971) in the form of spring and dashpot constants and equivalent mass of the soil. Karusum Keer and Lee (1966) presented an approximate analytical solution for the vertical and coupled horizontal and rocking vibration of an infinitely long rigid footing resting on the surface of an elastic half space and showed that coupling effects are quite significant.

The solution to soil-structure interaction problem is further studied by Kobori and Suzuki (1970) and Kobori, Minanui and Suzuki (1971) for visco-elastic layered medium and for a visco-elastic stratum of soil overlying a rigid base.

The effect of embedment of structures/footings has been recently considered. No rigorous analytical solutions of embedded footings are available because of the mathematical formulation. Barnov (1967) developed an approximate analytical approach based on the elastic half-space theory under the footing base and a series of thin elastic layers between the force surface of the ground and this half space. Beredugo and Novak (1972) considered the effect of coupled horizontal and rocking vibration of the same system studied by Novak and Beredugo (1972) for vertical vibrations. Krizek, Gupta and Parmelee (1972) to study the coupled and rocking vibration of a rigid strip footing embedded in an elastic stratum and made a parametric study of the effect of embedment.

There has been evidence that soil-structure interaction affects the response of structure to seismic response excitation. Several mathematical models are shown in Figs. 2 to 5 which have been used to take care of the foundation flexibility (1970, 1960, 1967). Gordon, Christiano and Epstein in their paper studied the influence of the layered media on the stiffness properties of foundation and proved that the thickness of the elastic stratum is not very important.

Further it can be seen that the difference between stiffness coefficients pertaining to circular and strip footings are important when selecting the corresponding parameter for actual foundations. Roesset and Wass (1974) proved that the static horizontal compliance varies more with layer
thickness than that of rocking compliance which converges rapidly to the half-space solutions as the layer thickness increases (i.e. \( H/B \rightarrow \infty \)). Also from their results it can be seen that the compliance functions (dynamic interaction coefficients representing the properties of the soil-structure for embedded footing) can be found approximately from those corresponding to a structure/foundation on the surface by using scaling factor.

Munirudrappa (1981) studied the response of the structures to soil-structure interaction by assuming the structure to rest on a flexible footing. The study has been made by introducing the interaction coefficients resulting due to an half space, assuming that the rock/hard surface is available at shallow layers. In detail, the response study has also been made with respect to embedment of the structure and proved that the interaction coefficients gets affected in turn reducing the amplitude of the structure.

Hence from the review of literature it can be seen that the response of structures/foundation to soil structure interaction is dependent upon the properties of the soil-stratum. Since the layered theory can be easily extended to the half-space theory with some scaling factors, results are presented based on the elastic half space theory, with reference to dynamic interaction coefficients. The results are presented in terms of horizontal displacement of the base mass, rotational displacement of the base mass, and flexural displacement of the top mass.

These are termed as the response parameters of the soil-structure interaction system of the building foundation system subjected to seismic excitation.

THEORETICAL ANALYSIS

The response of the structure is studied by an equivalent dynamic model having fewer degrees of freedom. Full scale structures of actual structures subjected to transient loading indicate, that the general method of replacing a complex structure by a suitable dynamic model yields satisfactory results for engineering purposes. Thus this study will utilise a mathematical model whose properties can be defined so that it will represent the dynamic response of any building in normal coordinates (Fig.1). This simple model has three degrees of freedom, horizontal translation of the base mass 'm', horizontal translation of the top mass 'm' and rocking or rotation of the base mass. To have the dynamic model compatible, some basic assumptions are made with respect to the foundation medium, the building model and
the seismic waves.

The basic equations of motions are
\[ \ddot{u} + \ddot{u}_b + \ddot{u}_\phi + 2\omega_r \dot{u} + \omega_r^2 z = -\ddot{u}_g \quad \ldots \quad (1) \]
\[ \ddot{u} + \alpha \ddot{u}_b + \ddot{u}_\phi + A_1 \ddot{u}_b + A_2 \ddot{u}_\phi + A_3 u_b + A_4 u_\phi = -\ddot{u}_g \quad \ldots \quad (2) \]
\[ \ddot{u} + \ddot{u}_b + \eta \ddot{u}_\phi + A_1 u_b + A_5 \ddot{u}_\phi + A_8 u_b + A_6 u_\phi = -\ddot{u}_g \quad \ldots \quad (3) \]

Where
\[ \alpha = \left[ \left( \frac{m}{m_b} \right) \right] \quad (4) \]
\[ \eta = \left[ \left( \frac{1 + b/2h}{2h} \right)^2 \right] \quad (5) \]

It should be observed that \( A_1 \) are obtained for steady state harmonic motion and are frequency dependent. Consequently Eqns.1 to 3 can only deal with harmonic excitation. The frequency dependant terms (Eqns.4 to Eqn.11) are nothing but the dynamic interaction coefficients given by Parmelee and Kudder (1973) based on the elastic half-space theory. These frequency dependant terms are given by
\[ k_T = \left[ \frac{1.68}{1.34 - \mu} \int V_s^2 \right] \quad \ldots \quad (6) \]
\[ d_T = \left[ \frac{7.66}{3.24 - \mu} \int V_s b \right] \quad \ldots \quad (7) \]
\[ k_{RT} = \left[ 1.41 \left( \mu - 0.55 \right) \int V_s b^2 \right] \quad \ldots \quad (8) \]
\[ d_{RT} = \left[ 0.565 \left( \mu - 0.55 \right) \int V_s b^2 \right] \quad \ldots \quad (9) \]

\[ k_R = \left[ \frac{1.58}{1.12 - \mu} \int V_s^2 b^2 \right] \quad \ldots \quad (10) \]
\[ d_R = \left[ \frac{0.50}{1.12 - \mu} \int V_s b^3 \right] \quad \ldots \quad (11) \]

These interaction coefficients can be modified with respect to embedment as
\[ k_T = \Delta k_T, \quad d_T = \Delta d_T, \]
\[ k_R = \Delta k_R, \quad d_R = \Delta d_R, \]

Where \( \Delta = e^{1.10\delta} \) and \( \delta = H/B \)

'H' represents the depth of embedment of structure or the thickness of the soil stratum. \( \ddot{u}_g \) represent the acceleration of the strong ground motion. \( u, u_b, u_\phi \) represent the response parameters of the structural foundation system. The solution to Eqns. 1 to 3 are obtained by the super position of 'H' harmonics and by the application of fourier transform techniques.
RESULTS, DISCUSSION AND CONCLUSIONS

The numerical computation for the solution of the problem has been carried out in IBM 360 and Dec-1090 computer of Indian Institute of Science, Bangalore. The response study has been made for different values of Poisson's ratio (μ), shear wave velocity (V), and embedment depth ratio (δ). Figs. 6, 7, 8 and 9 represent the response curves for shear wave velocity of 300 fps, 600 fps and embedment ratios of 0.0 and 0.2 respectively. Poisson's ratio (μ) = 0.00. Fig. 10, 11, 12 and 13 represent the behaviour of the response parameters for a single storey structure for different shear wave velocities against different values of embedment ratio. By referring to the work of Hoesset and Wass (1974), wherein the study has made with reference to dynamics of structures to layer media, it can be concluded from the comparison of their results with that of half space theory, found to be not much of variation. Hence the solution of the layered theory can be approximated to an elastic half space theory with some corrections.

From the detailed analysis of single-storey buildings (Figs. 10-13) considering the intermodal coupling it appears that the interaction of the structure-foundation system is significant only for shear wave velocities lower than 1000 fps. From the comparison of the response of the structure with embedment, it can be seen that the effect of embedment depicts the decrease in value of the response parameters of the structure-foundation system.

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Fig. 5. Discrete Representation of Layered Media.
ABSTRAIT

Le plan de fondation des structures pour résister aux chargements dynamiques à cause de tremblement de terre, doit rencontrer certain critère d'ouvrage. Ceux-ci racontent ordinairement à la reponse dynamique des structures exprimée en terme d'amplitude limitante de vibration à une fréquence particulière ou une valeur limitante de vélocité-maximum ou accélération maximum.

Dans cet article la réponse dynamique des structures supportantes sur ou enforcées dans une couche de sol est étudiée. Les cas divers analysés est au sujet des coefficients interactions dynamiques (des forces horizontales et des moments rupestres) résultante à cause des propriétés de la couche de sol et le mouvement à cause de l'accélération spécifiée de terre résultante à cause de mouvement fort de tremblement de terre. L'étude a été faite fondée sur le concept de Karasudhi, Keer et Lee (1966) et groupe Parmelee (1967). L'effet d'épaisseur d'assise et la convergence vers des solution à demi-espace sont recherchés. La réponse de la structure sans et avec l'enforcement est considérée pour évaluer la réponse de la structure à l'interaction de structure de sol.

On peut observer des résultats qu'on a obtenu que quelques uns des effets étudiés peut-être raisonnablement bien reproduits en appliquant facteur simple de correction aux solutions à demi-espace et tout à fait comparable avec celui du système assise. On a aussi remarqué des résultats que la vélocité de puffin, le rapport de poisson et l'effet d'enforcement auront une influence appreciable sur la reponse de la résultante de structure à cause de l'interaction de structure de osol.
SEISMIC ZONING OF INDIA

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ABSTRACT

Data on earthquake occurrence, damage during past earthquakes, geological and tectonic set-up and location of probable seismotectonic lineaments have been utilised by various workers for identification and demarcation of potential earthquake source regions and qualitative and quantitative evaluation of intensity of earthquake ground motion for preparation of seismic zoning maps of India. Such maps endeavour to portray the seismic hazard in different geomorphological and geological terrains in future. Average ground conditions in various terrains, pattern of earthquake occurrence, the type of buildings and other structures to be constructed, the available construction materials, the quality of construction commensurate with cost-benefit ratio limited by the economic and social considerations to provide adequate safety during the lifetime of the structures and to prevent severe disaster are the main governing factors in defining the level of earthquake resistance that should be provided in structures in various seismic zones. The various zones are assigned seismic design parameters governed by the estimated seismic hazard in terms of effective peak ground motion for a specified time duration. A seismic design zoning map of India, dividing the country in three seismic zones is recommended based on evaluation of seismic hazard from data on earthquake occurrence from 1917 to 1979, location of capable faults (faults with evidence of movements of Neogene to recent times and with association of estimable frequency-magnitude relation), probable focal depth distributions on such faults as well as floating events, and the attenuation of ground motion characteristics. Seismic design parameters have been evaluated for the three zones for earthquake resistant design of structures. The seismic design zoning map assumes average lithostratigraphic conditions in each zone, and do not consider the influence of local soil conditions, effects related to faulting in epicentral area and other local effects, which are incorporated in microzoning maps of specific regions.

ABSTRAIT

Les données sur des venues sismiques de dégâts produits par des tremblements de terre anciens, d'établissement géologique et de location des linéaments sismotectonique probables ont été utilisés par divers travailleurs à identifier et à délimiter des régions sismiques potentielles et à évaluer l'intensité du tremblement de terre qualitatif et quantitatif.
pour la préparation des cartes de zones sismiques de l’Inde. Ces cartes cherchent à faire sortir des risques sismiques dans de différents terrains géomorphologiques et géologiques dans l’avenir. Les conditions moyennes du sol de divers terrains, le type des bâtiments et d’autres structures à construire, des matériaux de construction, disponibles la qualité de la construction proportionnée à la raison de frais-profit bornée par des considérations sociales et économiques à fournir une sécurité suffisante pendant la vie des structures pour prendre des mesures préventives contre les accidents severes, sont des éléments principaux qui reçoivent la détermination du niveau de résistance sismique à donner aux structures dans des zones sismiques diverses. Ces zones sont attribuées des paramètres du type sismique réglés par des risques sismiques estimées en fonction du tremblement maximum effectif pour un temps spécifique. Une carte de l’Inde en zone selon des desseins sismiques, séparant le pays en trois zones sismiques, est proposée après l’évaluation des risques sismiques recueillies de données sur les venus sismiques depuis 1917 à 1979, la location des failles susceptibles (failles qui out fait preuve du mouvement de Neogène à Recent et sont liées avec une relation fréquence-magnitude estimable), des distribution de la profondeur focale probable sur de telles failles ainsi que des événements flottants et l’atténuation des caractéristiques du mouvement de terre. Les paramètres des desseins sismiques sont évalués de toutes les trois zones pour de types de structures qui résistent la secousses sismique. La carte en zones des desseins sismiques prend des conditions litho-stratigraphiques moyennes dans chaque zone et ne tient pas compte de l’effet des conditions locales du sol, des conséquences concernants les dislocations dans la région epicentrale et d’autres effect locaux qui sont incorporées dans les cartes en microzones des régions spécifiques.

INTRODUCTION

Seismology is a term used to describe the form and status of earthquake occurrence to indicate the probability and pattern of its recurrence in future, and its portrayal delineating areas of various levels of seismicity is called seismic zoning. Various attempts had been made for seismic zoning of India based on available information on earthquake occurrence damage during past earthquakes, geological and tectonic set up and location of probable seismotectonic lineaments and qualitative and quantitative evaluation of intensity of ground motion, and country was divided into various zones. The seismic zones demarcated by the Geological Survey of India (1935, in Auden, 1959) show three zones liable to severe, moderate and slight or nil damage and areas which suffered moderate to severe damage in past earthquakes with an intensity higher than Roeder-Furel intensity VII (Modified Mercalli Intensity VIII) within the severe zone (Fig. 1). West (1957) delineated three zones (Fig. 2) first danger zone liable to damage of a moderate to severe nature (with islands of epicentral tracts of past earthquakes covering areas of severe damage with Modified Mercalli Intensity X and above), second zone in which moderate damage may be caused by earthquake originating in the danger zone and severe damage close to epicentres within this zone, and third zone of comparative safety liable to slight or no damage. Krishna (1958, 1959) subdivided the Indian Subcontinent into four zones (Fig. 3) as areas with very heavy, moderate, and light damage and stable plateau region and computed ground acceleration due to an earthquake of magnitude 8 anywhere in very heavy damage zone and its southern margins along the frontal parts of the Himalayan belt at 0.3 g and above in heavy damage zone, 0.1 g to 0.3 g in moderate damage zone and less than 0.1 g in light damage zone. Mithal and Srivastava (1959) keeping in view the broad geotectonic framework of the subcontinent and data on earthquake occurrence demarcated three seismic belts (Fig. 4): belt of frequent earthquakes along Himalayan and Burman mountain belt, tectonic depressions and sedimentary basins bordering belt of occasional earthquakes along Nerbada, Tapti, Cambay and Kutch basins, Continental shelf region, Coastal tracts and region bordering belt of frequent earthquakes, and belt of few or rare earthquakes in the remaining parts of Peninsular shield. Such seismic zoning maps provided a qualitative schematization and thus served limited purpose for engineering applications.

INDIAN STANDARD DESIGN SEISMIC ZONING MAPS OF INDIA

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Fig. 1: Seismic Zones of Indian subcontinent compiled by Geological Survey of India in 1955 (with additions upto 1950; after Auden, 1959).

- Liable to severe damage.
- Liable to moderate damage.
- Liable to slight damage.

Area of moderate to severe damage in past earthquakes with an intensity higher than Rossi-Forel VII.

Fig. 2: Map of Indian subcontinent showing earthquake Zones by West (1957).

- Danger zone with epicentres of severe earthquake.

- Zone in which moderate damage may be caused by earthquakes originating in danger zone and severe damage close to epicentral tracts within the zone.

- Area of comparative safety.

- Areas of severe damage in past earthquakes.

Fig. 3: Seismic Zoning map of India subcontinent by Krishna (1958,1959) based on earthquake occurrence during the period 1904 - 1950.
In 1960, the Indian Standards Institution initiated action for the formulation of Indian Standard Recommendations for Earthquake Resistant Design of Structures (IS:1893-1962). This standard incorporated a seismic zoning map of India (Fig. 5) to indicate broadly the maximum expected Modified Mercalli Intensities. As the zoning map is included in the design standard, each zone specifies the level of design forces to be used to provide requisite earthquake resistance keeping in view the allowable stresses and strains in the structure, however it should be noted that the seismic coefficient used in the design of any structure depends on many factors and a rigorous analysis should be made in case of all important structures. Macroseismic intensity based on observed intensities during past earthquakes and evaluated from empirical attenuation relations was the main parameter (with minor modifications of iso-seismals for taking into account local geologic conditions and tectonic features, considered as possible active structures) for demarcation of the zones with the basic assumption that the geology of the region is the dominant controlling factor, both for intensities and frequency of occurrence of major earthquakes, and thus seismic zones based on intensity evaluation would be in conformity with earthquake occurrence, geomorphological conditions, geology and tectonics of the region, provided data on earthquake occurrence is available for long time duration. Iso-seismals were drawn for Modified Mercalli intensity V, VI, VII, VIII, IX and X and above delineating seismic zone I, II, III, IV, V and VI respectively (Fig. 5), and region with Modified Mercalli intensity "less than V" was designated a zone O. Design forces for each zone were prescribed based on engineering judgement. However as data on earthquake occurrence in Indian Subcontinent is not available for a long time period, the seismic zones in the 1962 map did not adequately represent the earthquake potential of the seismotectonic elements in which major earthquakes could occur in future and presented a picture with greater influence of past earthquake, specially the great earthquakes - 1819 Kutch earthquake, 1897 and 1950 Assam earthquakes, 1905 Kangra earthquake and 1934 Bihar-Nepal earthquake. With this in view when additional data on earthquake occurrence, geology and tectonics became available, the seismotectonic set up of the region was given greater significance in the revisions of the seismic zoning maps incorporated in the Indian Standard Criteria for Earthquake Resistant Design of Structures (IS:1893-1966 First Revision, IS:1893-1970 Second Revision, and IS:1893-1975 Third Revision). The 1966 seismic zoning map followed the same approach as for the 1962 map (Krishnaswamy, 1977) except that a greater recognition was given to the tectonic features.

The seismic zones in 1962 and 1966 seismic zoning maps of India were outlined mostly on data on past earthquake occurrence. In 1968 after the 1967 Koyna earthquake it was considered essential to give greater emphasis to the geotectonic setup so that the seismotectonic features as seismic sources in which earthquakes may occur in future are identified with genetic significance, and the estimates of intensity of earthquake are consistent with the geological cause, seismotectonic framework and the pattern of earthquake occurrence in each seismic zone. Thus taking into consideration the tectogenesis and geological history of the country and the possible nature of the operative processes which could lead to earthquake occurrence in future, the seismic zones were demarcated to broadly follow the areal extent and distribution of five principal seismotectonic units (Krishnaswamy, 1969 in Srivastava, 1974): the orogenic Unit of Cainozoic folding and faulting; the unit of Himalayan Fore deep and Marginal Depression; the unit of Peninsular Shield segmented by Tertiary/Quaternary fault movements, including the West Coast seismicogenic zone; the unit of Peninsular with Mesozoic fault movement and the later adjustments, including the Godavari graben and adjacent parts of the Shield as well as marginal parts of the Peninsular shield with platform cover of Mesozoic - Cainozoic sediments; and lastly the unit of generally stable Peninsular shield with locally partitioned areas bearing relatively ancient faults and localised seismic activity (Fig. 6).

Though a reasonable estimate of the possible maximum magnitude in each of the tectonic units could be obtained little information was available on depth of focus and it was difficult to establish definite associations of earthquake occurrence with tectonic features in these tectonic units. Thus possible maximum intensities around the tectonic features, based on known earthquake data, were mostly tentative, till evidence of movements along these feature during earthquakes could be confirmed. It was also considered desirable to reduce the number of zones from seven zones (including zone-O) to...
Fig. 4: Seismic belts of Indian subcontinent by Pittal and Srivastava (1958).

1. Seismic belt with frequent earthquakes.
2. Seismic belt with occasional earthquakes.
3. Seismic belt with few or rare earthquakes.

Fig. 5: 1962 Indian standard seismic zoning map (IS:1893 - 1962).

Fig. 6: Map showing generalised tectonic units of Indian Peninsular Shield and Himalayan region.

- Orogenic unit.
- Foredeep and marginal depression unit.
- West coast and Namada Chpti unit.
- Gondwana rifts unit.
- Shield unit.
- Faults.
in 1962 and 1966 maps to five zones in the 1970 map (Fig. 7), as the earthquake effects below Modified Mercalli intensity VI are insignificant in terms of design to call for a separation. Likewise the seismic zone with Modified Mercalli intensity 'X and above' was omitted as recorded strong ground motion characteristics in regions with Modified Mercalli intensity 'IX and above' show comparable ground accelerations, though with longer durations of strong ground motion with increase in magnitude.

**Probabilistic Approach**

The primary objective in the preparation of seismic zoning map is to prescribe design forces for earthquake resistant design of structures to ensure the desired safety of the structures. Average ground conditions in various terrains, pattern of earthquake occurrence, the type of building and other structures to be constructed, the available construction materials, the quality of construction commensurate with cost-benefit ratio limited by economic and social considerations to provide adequate safety during the life-time of the structures and to prevent severe disaster are the main governing factors in defining the level of earthquake resistance that should be provided in structures in various seismic zones. A rational basis is therefore required for prediction of design seismic forces due to the probable intensity of ground motion resulting from earthquake occurrence in future in various parts of the country and demarcate the seismic zones accordingly, since data on earthquake occurrence is not deterministic, it is appropriate to evaluate the ground motion parameters on probabilistic theory taking into consideration data on earthquake occurrence and available geological and geophysical information. Basu and Srivastava (1981) estimated 100 year effective peak ground accelerations at 2° x 2° degree grid points in Indian Subcontinent from probabilistic approach combining statistical and seismotectonic approach. The analytical model in this analysis considers the Indian Subcontinent to be nonhomogeneous in seismotectonic features. The Northern Himalayan seismic belt have been subdivided into three macro seismic provinces corresponding to Western, Central and Eastern Himalayan seismic provinces postulated by Srivastava et al. (1974), and Pre-Cambrian Peninsular shield is considered as one broad seismotectonic province. Seismically active structures (major active faults, Gondwana rifts, etc.) are taken as area sources within these broad seismic tectonic provinces. Seismic sources at every grid point is considered as volume source (for floating earthquakes) and area sources lying within the volume source. The magnitude, location and occurrence of earthquakes are considered as random variables and an attenuation law correlating peak ground acceleration with earthquake parameters for a source including the effect of scatter are used to estimate 100-year peak ground acceleration.

Following in similar lines of Basu and Srivastava (1981) 100-year effective peak ground acceleration were estimated at 2° x 2° grid points using data of earthquake occurrence from 1917-1979. Evaluation of damage and collapse possibilities of the structural system subjected to the ground excitation is necessary to construct seismic zoning map. A structural system is usually idealized as a multi-degree freedom system in which each natural mode of vibration behaves as a single degree freedom system. The elastic response spectra (spectral acceleration) give maximum response (absolute acceleration) of a set of viscously damped single degree freedom systems subjected to dynamic force or motion. The spectral acceleration tends to the peak ground acceleration as natural period of the structure tends to zero. The ratio of spectral acceleration and peak ground acceleration is known as amplification factor. Newmark and Hall (1969) have averaged spectral acceleration of several available strong motion data for various groups of ground conditions. They recommended median amplification factor 2.74, 2.12 and 1.37 for 2%, 5% and 10% respectively of critical damping for firm ground condition. The damping values 2%, 5% and 10% of critical damping roughly correspond to welded steel/prestressed concrete/reinforced concrete (lightly cracked), Bolted and/or riveted steel/reinforced concrete (highly cracked) and reinforced masonry structures respectively. The average spectral acceleration remains constant in the period below 0.03 sec, and is equal to effective peak ground acceleration. The amplification factor increases from 0.03 sec, which reaches a maximum value at 0.1 sec period, and remains constant above 0.4 sec period. Beyond 0.4 sec period, the amplification factor exponentially decays to zero as period becomes unbounded. The structural system are generally ductile capable of undergoing substantial deformation at constant loading without loss of strength in subsequent loading. The ratio of maximum relative displacement to the yield displacement is termed as
Fig. 7: 1970 Indian standard seismic zoning map (IS: 1893-1970 Second Revision; IS: 1895-1975 Third Revision).

Fig. 8: Spectral acceleration contour map. Spectral acceleration values are in cm/sec^2 for ordinary structures with 0.1 sec. to 0.4 sec. period.

Fig. 9: Seismic zoning map of India recommended for earthquake resistant design of structures.
ductility factor $\mu$. If small excursions in the inelastic range are permitted for an economic design, the maximum spectral acceleration is reduced by a factor $1/\mu$ in the period range (0.1 - 0.4 sec) of most of the ordinary structures, and taking into consideration allowable increase of permissible stress $\sigma_\text{T}$ as earthquake can be taken as accidental loading, the spectra are further reduced by $1/\sigma_\text{T}$, where $\sigma_\text{T} = 1 + \sigma_\text{E}$, to get design response spectrum. This total reduction factor from peak ground acceleration to the design spectrum is $r = \eta/\mu \sigma_\text{T}$, where $\eta$ is the amplification factor.

Only one strong motion record (1967 Kuyra earthquake) is available in Indian Subcontinent. It is difficult to obtain a generalised shape of the average spectra. A set of synthetic accelerograms (Srivastava and Daru, 1982) were therefore simulated for normalised peak ground acceleration of Igual for firm ground conditions. The elastic spectra of the each of the synthetic accelerogram were obtained, and averaged to get the shape of the spectra, considering the damping of various types of structures and respective ductility factor and increase of permissible stress as per Indian Standard Criteria for Earthquake Resistant Design of Structures (IS 1893-1975), reduction factors for design spectra were obtained as given in Table 1.

<table>
<thead>
<tr>
<th>Damping % of Critical ((\eta))</th>
<th>Amplification ((\mu))</th>
<th>Ductility Sable ((\mu))</th>
<th>Permissibility ((\mu))</th>
<th>Reduction Factor ((r))</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4.2</td>
<td>6</td>
<td>33%</td>
<td>0.52</td>
</tr>
<tr>
<td>5</td>
<td>2.75</td>
<td>4</td>
<td>33%</td>
<td>0.52</td>
</tr>
<tr>
<td>10</td>
<td>1.56</td>
<td>3</td>
<td>-</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Table 1: Values of Reduction Factors

The peak ground acceleration evaluated at various grid points of Indian Subcontinent were multiplied by the reduction factor to obtain average maximum design spectral acceleration of structures in the 0.1 - 0.4 sec period. The contours showing maximum average spectral acceleration were drawn and are shown in Fig. 8. The contour map shown in Fig. 8 was modified to take into account old historic earthquake data (prior 1917), presence of mobile belts along which tectonic basins exist in western parts of India and presence of tectonic lineaments (Marwara - 

Zone - Damodar belt, etc.) excluded from analysis due to absence of estimable magnitude frequency distribution. The zone factors were computed by averaging spectral acceleration within a particular zone and are given in Table 2. The seismic zone map is shown in Fig. 9.

Table 2: Value of Zone Factors for Basic Seismic Coefficient

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Zone No.</th>
<th>Zone Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>i)</td>
<td>III</td>
<td>0.1</td>
</tr>
<tr>
<td>ii)</td>
<td>II</td>
<td>0.03</td>
</tr>
<tr>
<td>iii)</td>
<td>I</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Table 2 gives the recommended design spectral acceleration for each zone, for structures 0.1 - 0.4 sec period. In order to evaluate the design spectral accelerations for periods less than 0.1 sec and greater than 0.4 sec, the shape of the average spectra in these range has to be modified. The ductility factor decreases linearly to unity (Newmark and Hall, 1969) as period of structure approaches 0.05 sec from 0.1 sec independent of structural type and remains constant below this period. The reduction factors of spectral response was obtained for all the period range and normalised with the reduction factor shown in Table 1 to obtain Average Design Spectral Factor (Fig. 10). The structures with period less than 0.03 sec are very rigid structure like rock structures (in case of jointed rock minimum damping may be 10% of critical) and the curves of 2% and 5% damping in this period range do not have any practical significance.
The basic seismic coefficient of a particular type of structure (damping) in a site is obtained by multiplying normalized average design spectral factor from Fig. 10 with the zone factor given in Table 2.

ACKNOWLEDGEMENT

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SEISMIC RISK EVALUATION FOR ENGINEERING PROJECT SITES

L’ÉVALUATION DES RISQUES SISMIQUES POUR DES SITES DE PROJETS CIVILES

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ABSTRACT

Data on earthquake occurrence, geological and tectonic set up of the area and evidences of movements in geologically recent times are utilised to identify seismotectonic lineaments capable of generating an earthquake. There is always some uncertainty associated with such data for evaluation of ground motion due to earthquakes, at structures, systems and components important for the safety, designed to remain functional during and after the earthquake. No unique methodology has so far been evolved to account for all the uncertainties in geological and geophysical data for estimating earthquake probability and evaluation of seismic risk for practical applications. Greater reliance is thus made on data on earthquake occurrence and strong ground motion records for prediction of ground motion with the assumption that the same pattern of earthquake will be valid for the future. In case seismological data is meagre or not available, extrapolation of information from regions of past earthquakes to potential source regions in the vicinity of the site is attempted for evaluation of ground motion. The schedule of investigations for evaluation of seismic risks at a project site and the methodology to account for various uncertainties associated with the available data for determining design forces corresponding to permissible risks, therefore requires an integrated deterministic (seismotectonic) and probabilistic approach for analysis, keeping in view economic and social considerations, in addition to engineering considerations. Special seismological, geological and soil investigations are required for nuclear power plants, river valley and other major engineering project sites and specific considerations are needed for establishing seismic design criteria for various structures.

ABSTRAIT

Les données sur des venues sismiques, l’établissement géologique et tectonique de la région et les évidences des mouvements dans le temps géologique récent, sont utilisées à identifier des linéaments sismo-tectoniques capables de produire un tremblement de terre. Il y a toujours, une certaine incertitude de telles données pour l’évaluation du mouvement de terre par la secousse sismique sur des structures, et des systèmes et des composants importants pour la sécurité destinée à rester fonctionnelle pendant et après le tremblement de terre. Une méthodologie unique n’a été développée jusqu’ici d’expliquer toutes les incertitudes de données.
Strong ground motion due to an earthquake could affect the safety of the structures at an engineering project site and consequent damage and loss of their functions, which may lead to disasters. Seismic risk evaluation endeavours to estimate the susceptibility to damage as well as the seismic hazard which may result in adverse effects at a site. Seismic risk is the probability of occurrence at a given site, within a given period of time, of potentially destructive seismic ground motion equal to, or greater than a specified intensity. Realistic evaluation of seismic risk depends on the available knowledge and ability to express the phenomena in physical terms suitable for analysis for understanding the origin and mechanism of earthquakes, identify seismic source regions, determine resulting time-history of strong ground motion due to maximum earthquake potential of the seismic sources, and evaluate the behaviour of ground and structures and other associated hazardous phenomena. Destructive earthquakes occur at relatively large intervals of time and their location, magnitude and time of occurrence can not be predicted accurately. Thus a large number of structures may not be subjected to extremely strong ground motion during their lifetimes and it may be considered adequate to adopt the design criteria for ordinary structures at a lower seismic hazard intensity, with the expectation that structures may experience repairable damage. However similar damage in dams, nuclear power plants, multi-storied buildings, long-span bridges, and other important structures lead to loss of function or collapse of structures during and after the earthquake and result in disaster. Special investigations are required for seismic risk evaluation for determining ground motion for design of such structures, and their components important for the safety, keeping in view permissible damage due to the maximum earthquake potential inside the seismotectonic province of the site and proximity of seismic sources from the site.

BASIC DATA REQUIRED FOR SEISMIC RISK EVALUATION

The data required for seismic risk evaluation at a site consist of information on earthquake occurrence and its effect, and information on geotectonic and geophysical features characterising the seismotectonic province of the site (and seismically active structures). If data on earthquake occurrence is available for a sufficiently long duration with desired quality and precision, it implicitly portrays and defines the seismotectonic province and detailed statistical analysis of such data, modified by relevant geological and geophysical information provide reasonable estimates of strong ground motion with the assumption that the pattern of earthquake occurrence in the past will repeat in future. Thus in addition to the data on earthquake occurrence, information required for evaluation of earthquake ground motion can be roughly divided into two categories.

A. Information relative to earthquake sources: about geological features of the region and about tectonic deformations, fault displacements, etc., for
identifying the active regions, the seismotectonic provinces and the seismically active structures and evaluation of their maximum earthquake potential.

B. Information relative to intensity of ground motion and propagation and attenuation of seismic waves, their amplitudes, frequencies, etc., for evaluating the ground motion at the site by the earthquake occurrences from such maximum earthquake potential at the nearest point to the site within the seismotectonic province of the site on the seismically active structure or at the borders of the adjacent seismotectonic provinces.

The data on earthquake occurrence form the basis of statistical analysis. Earthquakes are usually described giving date, origin time, epicentral coordinates (geographical latitude and longitude), depth of focus, magnitude and maximum intensity. Dimensions and orientation of rupture (fault), stress drop, seismic moment and spectral characteristics of recorded ground motions are also evaluated.

For major earthquakes reports on damage and other effects of earthquakes are prepared and isoseismal maps are compiled. The data on historical earthquakes often contain inaccuracies in dates, descriptions of the size of the earthquake, its effects and epicentre (or maximum intensity) locations. Reliable data on earthquake occurrence are usually not available for a sufficiently long period. For recent earthquake in the second half of the 20th century, the data is usually accurate, the accuracy varying according to the number, position and reliability of recording stations. The limitations of the degree of accuracy of the earthquake source location, defined in routine calculations as a point source (whereas seismic energy is released in faulting which may extend horizontally for a large distance, e.g., 500 km when the shock is very large with Richter magnitude greater than 8) often do not permit precise geologic inferences and identify seismically active geologic structures. The data on earthquake occurrence thus provides a first-order estimate of seismic hazard, which requires modification based on geological, geodetic and geophysical information.

Geological inferences about long term seismic activity are utilized to supplement the data on earthquake occurrence, and an effort is made to demarcate geographical, geological and geophysical features delineating the seismotectonic province appropriately bounding the source regions for future earthquakes. Characteristic structures in areas within such a seismotectonic province, where earthquakes are known to have occurred, are considered as seismically active structures. Major fault zones and crustal plate boundaries are considered as potential active seismic lineaments. However information about geologic history, movements during Neogene-Quaternary and Recent period, continuity of contemporary movements indicated by geodetic observations, geophysical anomalies (gravity, magnetic, geothermal, etc.) and young volcanism provide supporting evidence for identification.

MAXIMA EARTHQUAKE POTENTIAL AND GROUND MOTION

The maximum earthquake potential of a seismic source is generally considered to be indicated by the upper bound magnitude of earthquake likely to occur in future and is also referred as upper threshold magnitude \( M_{\text{max}} \) prescribing its long term seismicity. The value of largest possible magnitude limits the magnitude-frequency distribution of earthquake occurrence of the seismic source. Estimation of upper threshold magnitude from data on earthquake occurrence assumes that future activity will have similar magnitude-frequency distribution observed during the period for which data is available.

Such an assumption may not be valid if data is available for a short time duration (e.g., from a temporal seismic gap).
It is very difficult to estimate the upper threshold magnitude for regions of low seismicity.

The upper threshold magnitude for a seismic source is an important parameter for evaluation of ground motion characteristics at a site as a function of magnitude and focal distance from the seismic source. In the Himalayan belt of high seismicity, it is known that earthquakes of magnitude 6 or greater have occurred (1897 and 1950 Assam earthquakes, 1905 Kangra earthquake, 1934 Bihar-Nepal earthquake) and an upper threshold of 8.6 may be taken. Various approaches have been adopted for estimation of upper threshold magnitude for region of lesser seismicity by extreme value statistics (Cumal, 1958), by correlation of mean with seismic activity (Riznichenko, 1966) from the slope of magnitude-frequency graph (Vinogradov, 1962; Mogi, 1967; Scholz, 1968), from thickness of active layer (Shebalin, 1971), from length of seismic source (active fault), from fault offset during recent times and other criteria. In order to make use of all available geological and geophysical information connected with earthquake occurrence, pattern recognition method (Jain, 1974; Borisov et al., 1975), which estimate the influence of various parameters has been often used to estimate the upper threshold magnitude for a unit area.

Various ground motion attenuation relations based on statistical analysis of the ground motion records have been proposed by various investigators correlating ground motion parameters (peak ground particle acceleration, velocity displacement, etc.) with magnitude, distance from the seismic source and depth of focus to evaluate the intensity of ground motion at a site. These relationships established for different geological terrains in various countries, show a wide scatter of values for the same magnitude and distance (focal distance/epicentral distance). Fig. 1 shows a plot of attenuation of ground acceleration for magnitude 6.5 earthquake with nominal depth of focus for attenuation relations proposed by Ballard (1969), Donovan (1974), Esteve and Villaverde (1974),

Fig. 1: Attenuation of Ground Acceleration Observed in Kyna and Broach area and Computed by Empirical Formulae

Ballard (1975), Trifunac and Brady (1976) and McGuire (1977) and the attenuation as observed in Kyna region by Krishna et al. (1969) and Broach region by Krishna et al. (1971), which indicates that relations proposed by Ballard, Donovan, Esteve and
Villaverde and McGuire match with the attenuation observed in Deccan Trap region and relations proposed by Bath and Trifunac and Brady with the attenuation in Broach region (Cambay Basin) with Tertiary sediments and alluvial soil cover. For each site, it is therefore essential to determine how ground motion attenuates in the particular geologic terrain and appropriate attenuation relation established for evaluation of ground motion intensity.

Maximum macroseismic intensity assigned to the historical earthquakes, is often taken as a measure of maximum earthquake potential of the seismic sources in the region. Because of subjective judgment in evaluation of macroseismic intensity from observed damage to structures, geological effects, etc., for the desired precision and accuracy and also difference in construction and foundation conditions which govern the behavior of the structures, it is difficult to correlate observed intensities with ground motion parameters.

**DESIGN BASIS GROUND MOTION LEVEL**

The distance of the seismic sources, their magnitude frequency distribution, depth distribution and attenuation relation have to be considered to evaluate the intensity of ground motion at a site. The methodology is directed towards formulating a rational model to estimate the ground motion, using either deterministic or probabilistic assumptions. The deterministic evaluation, also called as seismotectonic approach, do not give frequency of occurrence and the probability of exceedance of the computed intensity of ground motion, and represent a very conservative estimate of seismic hazard for the site.

The important consideration in evaluation of seismic risk and evaluation of design forces is the consequences of damage of a particular type of structure from shaking, from ground failure etc. Well designed and constructed structure will be less susceptible to damage than old, poorly constructed structure. Very important structures, such as dams and nuclear power plants, whose failure would lead to disaster from secondary phenomena, should be provided with least susceptibility to damage than ordinary masonry buildings, which may be permitted to undergo repairable non-structural damage but prevented from structural failure and collapse. Thus the design criteria for very important structures will be different from that for ordinary and/or conventional buildings. Engineering project site with such structures therefore require special considerations in defining appropriate levels of severity of ground motion at a given site to permit analysis of the behavior of the structure designed to remain functional during and after the earthquake. One of these levels of severity of ground motion for design could be considered to be the maximum ground motion which reasonably can be expected to be experienced at the site once during the life time of the structure. The other level may correspond directly to ultimate safety requirements. This level of ground motion has a very low probability of being exceeded and represents the maximum level of ground motion on the basis of estimates of upper threshold magnitude of seismic sources. These two levels of severity of design basis ground motion are referred SI and S2 by the International Atomic Energy Agency (I.A.E.A. safety guide No. 50-36-S16, 1979).

The SI is derived on the basis of historical earthquakes that have affected the site, expressed as ground motion having a defined probability of not being exceeded and may be derived using probabilistic approach or the approach may include seismotectonic consideration (combined probabilistic and seismotectonic approach). An alternative to rigorous probabilistic analysis for evaluation of S1 when data on earthquake occurrence is meager or not available S1 is taken as a fraction (e.g. one-half) of S2, where S2 is determined by rigorous application of a seismotectonic method.

The S2 derived on the basis of maximum earthquake potential inside the seismotectonic province of the site or adjoining seismotectonic provinces associated with or not associated with specific tectonic structures, and a combined probabilistic and seismotectonic approach may also be used based on available data on earthquake occurrence. For a site for which seismotectonic data is available IAEA has recommended that S2 evaluation may be performed as given below:

1) "For each seismically active structure the maximum earthquake potential should be considered to be moved to the appropriate location on the structure closest to the site area. For earthquakes near to the site the physical dimension of the source may be taken into account."
(2) "The maximum earthquake potential in the seismotectonic province of the site that cannot be associated with seismically active structures should be assumed to occur at a certain distance from the site... This distance may be in the range of a few to tens of kilometers and depends on the focal depth of earthquakes of the province. In evaluating it the physical dimension of the source will also be considered".

(3) "Maximum earthquake potential in seismotectonic provinces adjacent to the province of the site should be assumed to occur at the locations on the province boundaries nearest to the site".

(4) In case "seismotectonic data are inadequate" and the procedure described in 1, 2 and 3 cannot be used, "the maximum earthquake for the region is determined. The S2 is then defined as one or two units of intensity (MM or Modified Mercalli scale) more than the value of the maximum historical earthquake. In areas where the historical earthquake record is short, this procedure to estimate the S2 may give results which are not reliable".

Methodology and techniques for the probabilistic estimations of ground motion have been developed by various workers (Cornell, 1968, Esteva, 1968, Algermissen and Perkins, 1972, McGuire, 1975 and Basu, 1977 etc). The combined statistical and seismotectonic approach for evaluation of ground motion parameter (acceleration, velocity, displacement) at a site involves identification of seismotectonic province of the site and seismic sources in which future significant earthquake can originate, determine the rate at which earthquake can occur in different sources, obtain the frequency distribution of depth of focus and magnitude in various sources, and establish a ground motion attenuation to account for the effect of focal distance of earthquake on the site. The analysis is carried out with the assumption that the available data is not exhaustive and contain error in locations (say 0.1 degree), depth estimates, magnitude etc. The statistical tool available for analysis is Bayesian analysis, and can be carried out on the lines of Basu (1977) for evaluation of ground acceleration in following steps.

(i) The data are sorted out for different seismic sources.

(ii) A modular source of arc length 150 km at the surface of the earth with the project site as its centre and of 150 km depth is taken and seismically active faults lying within modular source are considered as area sources. Location of floating earthquakes (earthquakes not associated with seismically active structures) in the modular source are considered temporarily stationary and spatially homogeneous, and occurrence of earthquake is equally likely in the latitude and longitude direction. The focal depth data assigned a value of 35 km (average depth of Moho) are assumed to be distributed uniformly within 16 to 51 km for estimation of local depth distribution. A mixed truncated lognormal distribution is fitted in the modular source and area sources. The probability density function of focal depth is
\[ f_R(H) = \sum_{i=1}^{2} p_i \exp \left[ -\left( \ln H \right)^2 / (2 \nu^2) \right] / \left\{ \sqrt{2 \pi \nu^2 \ln H_0} / \sqrt{v} \right\} \] ... (1)

where,

\[
p_1 < p_2 < 1
\]

\[
\nu = 1 - p_1
\]

\[
\nu_1 > \nu_2 > 0
\]

\[
h \in [0, H_0 = 150 \text{ km}]
\]

and \( \mathcal{G}(\cdot) \) probability distribution function of \( N(0,1) \). The estimation parameters \( (p_1, \nu_1, \nu_2, \gamma) \) is formulated as a minimum chi-square problem.

The magnitude, \( M \), of earthquake is to be independent of the occurrence of earthquakes. The magnitude distribution is estimated in two ways in accordance with available data. A bilinear frequency magnitude relation with an upper and lower threshold is used. This probability density function is

\[
f_m(m) = k_2 \beta \exp \left[ \left( m - m_2 \right) \beta \right] m \in (m_0, m_2)
\]

where,

\[
1/k_2 = \left[ \lambda + \beta - \lambda \exp \left\{ \left( m_0 - m_1 \right) \beta \right\} - \beta \exp \left\{ \left( m_0 - m_2 \right) \beta + \left( m_1 - m_2 \right) \lambda \right\} / (\lambda + \beta) \right] \] ... (2)

and \( m_0 < m_1 < m_2 \).

The \( U (\cdot) \) is a Heaviside function. It is assumed that \( m_0 = 5 \log (\text{erg}) \) is the lower threshold magnitude. The estimation of parameters \( (\lambda, \beta, m_0, m_2) \) are obtained using minimum chi-square estimation method. Physical interpretation of Eq. 2 is that there exist two different processes one leading to release of energy below magnitude \( m_0 \), and other above that. The upper threshold magnitude \( m_2 \) takes care of the ultimate strength against rupture of earth's crust in upper mantle. In case of small data the upper threshold \( m_2 \) is assumed from historical and/or geological data and linear frequency magnitude relation with upper and lower threshold is used. The probability density function is

\[
f_m(m) = k_2 \beta \exp \left[ \left( m - m_2 \right) \beta \right] m \in (m_0, m_2)
\]

where,

\[
1/k_2 = 1 - \exp \left[ (m_0 - m_2) \beta \right]
\]

The attenuation law correlating peak ground acceleration with earthquake parameters for a source including the effect of scatter in the past data is assumed to be

\[
Y = a \exp \left[ bM - c \ln (R + d) + e \right] \] ... (4)

where \( M \) is the magnitude in \( \log \) (erg), \( R \) is the focal distance in km, \( Y \) is the peak ground acceleration and \( \epsilon \) is a normal random variable. Esteva and Villalve (1974) have suggested values of \( a, b, c \) and \( d \) as 5600, 0.8, 2 and 40 respectively and normal random variable \( \epsilon \) is of mean 0, 0.04 and variance 0.0496.

The occurrence of an earthquake is assumed to be in accordance with Poisson process with intensity \( \mu_i \) for magnitude greater than five. The posterior intensity of earthquake arrival is estimated through Bayesian statistics and by using past regional (Newmark and Rosenbluth, 1971) seismic data. Under the assumption of mutual statistical independence of various sources at project site, it can be shown that, the probability distribution function of maximum peak acceleration, \( Y_{\text{max}} \), can be shown to be

\[
P_{Y_{\text{max}}} (y, t) = \exp \left\{ \sum_{i=1}^{n+1} \mu_i \left[ y_i > y \right] \right\} \] ... (5)

in which \( \mu_i \) is the intensity of earthquake arrival in ith source, \( n \) is the number of active faults in modular source at site and \( P [y_i > y] \) is the probability of exceeding peak acceleration \( y_i \) at the site due to the ith source. From Eq. 5 t-year return period of peak ground acceleration, \( y \), at the site is obtained as

\[
t = 1/\left\{ \sum_{i=1}^{n+1} \mu_i \left[ y_i > y \right] \right\}
\]

Substituting \( t \) numerically the t-year acceleration is obtained at the site solving Eq. 6 numerically for \( y \).

In general peak acceleration (velocity, displacement) for 100 years service life of the structure for various exceedence probability are evaluated from Eq. 5. A low exceedence probability is taken for atomic power plant structures (say 0.05). For dams less conservative exceedence pro-
bility (say 0.25) may be taken. The choice of the ground motion parameter with prescribed exceedence probability is made on an engineering judgement based on permissible damage to the structures and prescribing the levels of design ground motion.

**DESIGN EARTHQUAKE MOTION**

The safety of the structures in a project site subjected to ground excitation is determined from the structural response. The structural system is idealized as a multidegree freedom system, in which each mode of vibration is a single degree freedom system. The spectral acceleration (spectra) is a representation of maximum response of a set of linear-elastic-damped single degree freedom systems subjected to ground motion in frequency (period) domain. In case of availability of strong motion record within the seismotectonic province of project site, the shape of spectra may be assumed of similar nature. However, frequency scale may be modified in the ratio of S-wave velocity to suit geological site condition of project site. The ensembles of time history ground motion can be simulated by matching modified spectra for the site [Khal, 1972; Olski et al, 1979]. However, the strong motion record are often not available. This fact makes it necessary to simulate earthquake ground acceleration consistent with tectonic set up of a project site. A correlated noise is simulated with one sided power spectral density function

$$C_X(f) = \frac{3}{(\omega_c^2 + 4\pi f^2)^\lambda} \left[ 1 - 2 \exp (-\omega_c^2) \right] \cos(2\pi t_c f) ; f \geq 0$$  \(\text{(7)}\)

where $3 = 4\lambda (P_a S_a)^2 / T_0$, $P_a$ = maximum peak ground particle acceleration, $S_a$ = maximum spectral acceleration amplification ($1.5$ for rock and firm ground and $1.6$ for alluvium), $T_0$ = duration of ground motion, $\lambda = 1 + \exp (-2\lambda \omega_c^2)$, $\omega_c$ = finite correlation time = $1.33$ sec and $\lambda$ = parameter of exponential decay correlator = $1.5$.

The maximum amplification of spectral acceleration as suggested by Khanna et al. (1977) is chosen according to the particular site conditions (rock and firm ground, or alluvium). The maximum peak ground particle acceleration is assumed to be constant. The duration of ground motion is dependent on the magnitude of the shock and is obtained using Bolt (1974) and assuming that strong motion duration is equal to rise and decay duration

$$T_0 = 35 \tanh (M - 6.5) + 38$$  \(\text{(8)}\)

where $M$ is the upper threshold magnitude. The correlated noise is simulated in the frequency domain by

$$\begin{align*}
X(t) &= \int_0^\infty \pi \sum_{k=1}^{\infty} A_k(r_k) \cos \left[ 2\pi (f_k t + \Theta_k) \right] \, dk \\
\Theta_k &= \text{uniformly distributed random variable in the range } (0, 1). \text{ Hence the coefficient } A_k \text{ is given by}
\end{align*}$$

$$A_k(r_k) = \sqrt{G_X(r_k) \Delta f}$$  \(\text{(10)}\)

The resulting noise is modulated in the time domain by a modulator

$$M(t) = (t/	ext{tm})^d \exp \{ - \alpha (1 - t/	ext{tm}) \}$$  \(\text{(11)}\)

where $\alpha$ and $\text{tm}$ depend on the upper threshold magnitude of the earthquake. The generated motion $V(t)$ is passed through a filter

$$\ddot{y} + 2\xi \omega_p \dot{y} + \omega_p^2 y = - V(t)$$  \(\text{(12)}\)

where $\xi$ and $\omega_p$ depends on the ground condition. The ground acceleration $\ddot{y}(t)$ is further base line corrected. The corrected motion is modified between $0$ to $1$ second such that at starting time ground motion is zero and at one second is equal to the base line corrected motion. The resulting modified motion is normalized to make peak ground acceleration equal to the design basis level. The parameter $\xi$ and $\omega_p$ is chosen to match the period at which amplification of spectral acceleration would be maximum according to the site conditions.

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THE SENERCHIA LANDSLIDE TRIGGERED BY THE 23 NOVEMBER 1980 EARTHQUAKE

LE GLISSEMENT DE TERRAIN DE SENERCHIA CAUSE PAR LE TREMBLEMENT DE TERRE DU 23 NOVEMBRE 1980

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ABSTRACT  The role played by earthquakes in landslide triggering could be of tremendous importance in evaluating seismic risk and/or protecting many regions of Italy.

After defining the geomorphological and geological setting of the Sele river valley the main features of the Senerchia landslides are presented and discussed.

A parametric back analysis was carried out before the event, and then, once the operative geotechnical parameters were obtained, a pseudo-static analysis simulating the event was performed.

Numerical results confirm the geomorphological indication. The earthquake was simply a catalyst for the landslides since the slope was in a limiting conditions.

However it must be stressed that even if the slope had been completely drained there could have been a landslide anyway.

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1 - INTRODUCTION

In the evening of 23 November 1980, a $M = 6.8$ earthquake hit Irpinia (South Italy). The epicenter was at $40^\circ 46'N - 15^\circ 18'E$ (i.e. near Laviano, Santomenna and Castelnuovo di Conza, Fig.1); the depth was some 18 Km; the source was a dipslip normal fault with Apennine (NW-SE) strike and vergency towards the Tyrrhenian Sea (at SW) (Progetto Finalizzato Geodinamica-Gruppo di Lavoro Terremoto del 23.11.80, 1981). This event fits well into the frame of the seismic history of the area: similar events of 9 - 10 MSK degree had happened in Irpinia in 1930 and in 1962. The latter reached degree 10 in the above mentioned villages and in other villages just NW of the area in Fig. 1.

Many strong motions of the CNEN-ENEL seismic network were operating during the earthquake (CNEN-ENEL, 1980); from the accelerograms so far available we can evaluate; an acceleration of 0.33 m/sec$^2$ at Sturno, near the epicenter and a relatively high acceleration of 0.22 m/sec$^2$ at Brienza 44 Km from the source.

Among other disastrous effects, the earthquake triggered a number of landslides, some of which of huge dimensions, over a wide area; all these landslides were not new events, but resulted from the reactivation of old landslides, some of which were in "quieting" conditions since pre-historic ages. Only some rock falls, which are very common in the many rocky cliffs of this area, can be considered as newly generated events (Agnesi et al., in prep.). Very close to Senerchia (Fig. 1), a large landslide started to collapse some three days after the earthquake, that is in the same time span as the other major landslides triggered by the same. The landslide rapid (26.11-10.12), reached

Fig.1 - Geological-structural sketch of the upper portion of the Sele River Valley. The N-S side of the area is 25 Km long. Legend symbols: 1 - Quaternary marine and continental sediments, including well developed talus. 2 - Upper Cretaceous - Miocene allochthonous complex (Argille Varicolori). 3 - Mesozoic carbonatic allochthonous complex. 4 - main fault. 5 - main fault-scarp. 6 - fault reactivated by 23.11.1980 earthquake. Note as the graben is far from simple. Main fault-scarps are related to different sets of faults, among which the Apennine one (NW-SE) is predominant.

a length of some 2,200 m, a width of some 550 m and affected the slope down to a mean depth of some 45 m. Senerchia is as far as 11 Km from the epicentre area, and the MSK intensity reached the 8th
degree.

The role played by earthquakes in landslide triggering and/or in reducing the safety factor of slopes, is still a matter of argument, the results of which could be of tremendous importance in evaluating seismic risk and/or protecting many regions of Italy. Therefore, one of the aims of this paper is to contribute to the knowledge of these mechanisms by attempting to assess the combined effects on the stability by the earthquake shaking and other parameters (such as cohesion, friction angle, unit weight, water table depth) whose variation beyond limit-values could cause the triggering of the landslide.

A parametric back analysis was carried out before the event, and then once the operative geotechnical parameters, were obtained, a pseudo-static analysis after the event was performed, with the purpose of evaluating the minimum acceleration value required to trigger the landslide.

2 - GEOMORPHOLOGICAL AND GEOLOGICAL SETTING OF THE SELE RIVER VALLEY

The River Sele flows from the Campanian Apennine towards the Tyrrhenian sea (Fig. 1). In this part of the Apennine range, basins are characterized by a marked antecedence of drainage pattern, subsequently adapted to the structural trends of the latest tectonic up-lifting phases (Baggiioni-Lippmann, 1981); this way, the present main stream of the Sele flows from North toward South, while the main tributaries have a NW-SE attitude.

The upper portion of the R. Sele flows between the Picentini Mts and the Mt Marzano - Mt Ogna Plateau (Fig. 1), which are marked by high cliffs of tectonic origin. Indeed, the Mounts and Plateau are horst, and the valley-bottom is a graben between them. This tectonic style, resulting in rather complicated tectonic patterns (Fig. 1), is due to various up-lifting phases, that have affected all geologic units, that underlie the Apennine range, since the Upper Miocene. The latest one of these phases, is still active to-day, as witnessed by the seismic character of the area, indeed even the faults reactivated by the last event, are normal faults (Fig. 1; cf. also Bollettinari & Panizza, 1981, and Cinque, Lambiasi & Sgroso, 1981).

The geological units outcropping in the Sele River Valley can be synthetically put into two main groups:

- Allochthonous complexes
- Autochthonous complexes

The former are constituted of a rather complicated nappe building emplace during an Upper Miocene overthrusting phase (Ortolani & Torre, 1981). One can recognize two main complexes: a structurally lower complex made mainly of carbonates (Mesozoic to Oligocene) and an over-lying clayey-sandy-carbonatic complex ("Argille Varicolori") overthrust upon the other during an earlier (Lower Miocene) tectonic phase, and involved in the subsequent Upper Miocene phase together with it.

The latter group consists of sediments deposited from the Upper Miocene to Quaternary in marine and continental environments and are made of sands, sandy clays and silts, conglomerates.

Of course, the actual geology is rather more complicated with respect to this over-simplified description.

By the interplay of differential erosion and of neotectonic activity, the lowest and harder complex is now generally morpholo-
gically dominant over the overlying and softer layer which are still preserved in grabens (as the Sele River Valley bottom) or in depressions on the block faulted carbonatic plateaus (Fig. 1). Karst erosion deeply attacks the limestone levels of the carbonatic complex which also make important fresh-water reservoirs.

Also due to different past climatic conditions, thick layers of talus cover the foot of cliffs at the horst-graben boundaries; these taluses contain paleo-soil levels that indicate their multicyclical genesis; slope dismantling phenomena acting at present are mass-movement of various types on valley-bottom earth slopes and on rocky cliffs; some of these landslides are of huge dimensions, and even the largest ones triggered by the 1980 earthquake, are a reactivation of part of older landslides, except the Senerchia landslide which occurred at the boundary between a thick (more than 90 m) pseudo-terrace made of talus and the "Argille Varicolore" here represented by a clayey flysch sequence. Actually, the landslide originated on the "Argille Varicolore", while the talus has been involved by secondary slab failures of noteworthy dimensions (some 10 m thickness); this talus-made dominant feature, is due to a huge limestone block probably unrooted, at whose shoulders the debris has accumulated being preserved against erosion (Fig. 3); counter-slope dip of talus layers seems to indicate lateral spreading of the block and the talus.

3 - THE SENERCHIA LANDSLIDE

The landslide started moving as a well individualized slide-flow, three days after the earthquake, but already the morning after, some cracks were detected by people on the asphalt of the road leading from Senerchia to Oliveto Citra, passing near the aqueduct whose source is near the village, to the South (Fig. 2). During this first phase, until the aqueduct was switched-off, a large amount of water, of the order of some hundreds of cubic meters, entered the slope. On the other hand, the slope was in itself waterrich, due to the rains which had fallen until some eight days before; moreover, there is a seepage of groundwater from the carbonatic massif towards the layers of "Argille Varicolore" that have an even lower permeability due to jointing and sandy layers. This leads us to assume the water table level near or coincident with topographical surface.

The movement began as a translational earthslide (Skempton & Hutchinson 1969, Varnes 1978) rapidly evolving towards an earthflow. Some few days after the whole slope was intersected by deformations of different types and intensity. Indeed, as detected by field inspection in first days of December '80, and as later observed in aerial photos of 10.12. 1980, part of the landslide was still moving as a slide, with the hummocky surface still supporting trees, while most of the body was flowing at a relatively high speed (meters/day). A mud-flow (Hutchinson & Bhandari, 1971) some 400 m long, took place on the right side of the landslide (Fig. 2) probably because of water concentration; on the opposite site the picture was more complicated: slab failures affected the talus layers near Senerchia, while downslopeward translational slides and topples affect the sliding slopes with a movement direction slightly departing from maximum declivity direction, because of a transverse-slope com-
ponent caused by stress relief along the main landslide boundary (Fig.s 2 and 3). At the toe, the widespread development of cracks with counter-slope shear surface, along with ground bulgings, indicated the possibility of underthrust (Varnes, 1978) action in a softer layer at a few meters depth probably fashioned by a precedent flow. This situation regard the left side of the toe; at the right side of the landslide a flow tongue was forming that kept on flowing until the following summer not involving the parent slope any more, but just flowing over it.

The water richness of the landslide, was clearly shown by the large amount of surface water released by the landslide body; this wildly flowing water eroded new gullies in a few weeks and several ponds formed in the depressions (Fig. 2). 

By comparing maps derived from 1955 aerial photos, with maps derived from 10.12.1980 aerial photos, it is possible to individualize a waning area upslope and a waxing area downslope (Fig.s 2 and 3). Topographical differences permit us to evaluate the thickness of slab failures on the talus, in some 10 meters, and the general bulding of the left side of the toe.

The shear surface is exposed only at places, near the boundaries and at the main crown; nevertheless, the thickness of the material involved could be inferred waxing referred to the same contour line. 13 - Present (solid) and former (dashed) contour line. 14 - Remoulded sample. 15 - Aqueduct. 16 - Track of profiles in Fig. 3. The contour lines are only indicative due to lack of precision of the former ones obtained from very different scale maps (1:5,000 for the present and 1:25,000 for the former ones).
Fig. 3 - Along-landslide (A-A') and cross-landslide (B-B' to F-F') profiles. Legend symbols: 1 - Talus. 2 - "Argille Varicolori". 3 - Limestone block. Note the emersion of shear surface at the main crown, and the general lowering of the slope declivity.

by field inspection and from cross-profiles (Fig. 3), obviously basing our estimate on previous experiences. This way, one can expect a thickness of some 45 m at mid-length, where the landslide seems to have advanced because of slope overloading caused by the overriding material.

It should be stressed that the slide evolved downslopewards over some 2,000 m in a few days, and this suggests that the slope was highly susceptible to collapse. The largest displacements occurred along the "stream-like" flow, where a limestone block was dragged over a distance of some 900 m.

Notwithstanding the complexity of the collapse mechanisms, the shear surface must be rather regular, and it is most probably controlled by pre-existing slide or flow surfaces.

In spite of the huge amount of involved material (about $3 \times 10^7$ cubic m) and of the strong topographical modifications (up to 20 m in site lowering and up to 10 m in site raising; about 1° in declivity diminishing), the morphology of the area has not changed its style. This, along with morphological evidence detected by before-event aerial-photos, confirms that the slopes of this area are shaped essentially by mass-movements. (cf. also Baggioni-Lippmann, 1981 and Agnesi et al., in prep.). Moreover, and in accordance with the former statement, this landslide is a reactivation of an older event, which has been in quiescent conditions for a long time. Indeed, in the 1955 aerial photos it is not possible to recognise a well defined landslide body.

4 - STABILITY ANALYSES BEFORE THE EVENT

Only on the occasion of the 2nd National Congress on clays in 1976 (A.I.P.E.A. - Gruppo Italiano, 1977) were systematic studies begun in Italy on the geotechnical characterisation of the geological olistostonic formation of the Argille Varicolori, concerning the landslide. The subject was further investigated during the International Symposium on the Geotechnics of structurally complex formations in 1977 (A.G.I., 1977).

From the results so far avai-
able we can observe the mainly pelitic nature of the formation, with a sand-clay ratio of less than 1, and a complex structure of disarranged layers of clay or clay shales, after with a chaotic structure and with complexity depending on heterogeneity, mineralogy composition, and stress history of repeated cycles of shearing with large displacements (A.G.I., 1979).

From a general point of view the Argille Varicolore formation shows index properties of IL = 30 - 60, Ip = 20 - 40, and extremely variable shear strength parameters $c' = 0 - 10 \text{ KN/m}^2$, $\phi' = 10 - 23^\circ$. Owing to this variability, which is linked to the type of this formation's structural complexity, and as no specific investigation at the Senerchia site has been carried out, 7 samples of remoulded soil (Fig. 2) were taken and given identification and direct shear strength tests.

From the preliminary results of remoulded samples we obtained the following mean index parameters: clay fraction 39%, LL = 63, IP = 36; and strength parameters: $c' = 20 \text{ KN/m}^2$, $\phi' = 14^\circ$, $\phi' r = 9^\circ$.

On the one hand we had the difficulty of determining representative geotechnical parameters of the type of structural complexity of the formation in the laboratory. On the other hand there were the uncertainties in defining the exact profiles before the event, the sliding surface and the piezometric level. So to overcome these difficulties the stability analysis was carried out in parametric form, checking the influence lines on the safety factor ($F$) of the main geotechnical parameters before the event (Fig. 4).

The numerical investigation was effected by assuming as a reference point the parameter values: $c' = 15 \text{ KN/m}^2$, $\phi' = 15^\circ$, $\gamma = 1.9 \text{ t/m}^3$; moreover the piezometric level was assumed as mentioned before.

![Fig. 4 - Influence lines of geotechnical parameters and piezometric level on the safety factor $a = \text{piezometric level depth. } z_s = \text{mean depth of sliding surface.} $](image)

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to be coincident with the topographical surface.

From the results of Fig. 4 it is possible to note the considerable influence on stability of the piezometric level as expressed in an adimensional ratio between the piezometric level depth \( z \) and the sliding surface depth \( Z_s \), both referred to the topographical surface.

An especial mention is given to the line of influence of the specific weight. This line and the angle of friction influence line are slightly curvilinear, whereas the cohesion \( c' \) and adimensional piezometric level \( a/Z_s \) influence lines are rectilinear.

As an exception to the general rule, the increasing weight of the sliding body here has a positive effect on stability. This might be related to the fact that the angle of friction \( \phi' \) is greater than the angle of the sliding surface which is \( \theta^\circ \), and so the positive effect of the major friction strength is greater than the negative effect on a partial safety factor due to the slight cohesive component.

All the same it is evident from Fig. 4 that the variation of \( \gamma = 1.7 - 2.2 \, t/m^3 \) has not as much influence on \( F \) as the variation of other geotechnical parameters.

In Fig. 5, once assuming \( \gamma = 1.9 \, t/m^3 \) the line of influence on possible couples of strength parameter values \( c' \) and \( \phi' \), has been calculated. In Fig. 5a we show the case of the piezometric level coinciding with the topographical surface.

From the graphs of Figs. 5a and 5b once we assume the strength parameter values from the laboratory \( c' = 20 \, KN/m^2 \), \( \phi' = 14^\circ \) we obtain \( F = 0.99 \) (5a) and \( F = 1.9 \) (5b).

5 - STABILITY ANALYSES AND DISCUSSION

The results of back analyses before the event shows some agree-

![Graphs showing influence lines of cohesion and friction angle on the safety factor.](image-url)

Fig. 5 - Influence lines of cohesion and friction angle on the safety factor.
ment between in situ shear strength and those detected by direct shear tests on remoulded samples (cf. Mauger, 1980).

Of course other piezometric levels, between the extreme conditions considered in Fig. 5, could be realistic. As an example we quoted F = 1.05 for a mean piezometric level depth a = 3 m.

The upper part of the slope is in a worse stability condition than the lower part; indeed numerical calculation shows the partial value of F = 0.90 before the event, while the overall value is F = 0.99 as said before.

That could be only partially balanced by a greater piezometric level depth on the upper part of the slope, which in any case could have moved independently in the past from the lower part, as there is some evidence from careful consideration of the topographical surface of the slope before the event.

The reconstruction of the above is fairly reliable since it is based on the examination of maps aerial photos before the events (cf. par. 3); whereas the sliding surface has been estimated. However the stability analysis for a sliding surface depth ranging between 35 and 55 m shows a slight variation in the safety factor within the 4% limit.

The seismic coefficient K, expressed as percent of gravity g is unknown. While waiting for attenuation law to be evaluated, in order to extend to the site the acceleration programs recorded by the seismic CNEN-ENEL network, in Fig. 6 a parametric analysis is shown for the influencing lines of the coeff. K on safety factor, for cases a/z_s = 0 (Fig. 5a) and a/z_s = 1 (Fig. 5b).

The Bell (1968) method was used for the calculations. This assumes the seismic force as horizontal. As a rule this is not correct as a preventive research of the worst direction should be made, which is usually different from the horizontal one (Maugeri and Motta, 1980). However the mistake diminishes with the diminishing cohesion, and is negligible for the cases:

Fig. 6 - Influence lines of seismic coefficient K on the safety factor. K = seismic coefficient in percent of g.
c' = 0 and c' = 20 KN/cm² as in Fig. 6.

From Fig. 6a one can see that the slope is in limit equilibrium and therefore might move even without and earthquake. From Fig. 6b a minimum seismic coefficient K = 0.15 is required to trigger the landslide.

On the basis of the acceleration value reported in paragraph 1, and the basis of empirical correlation (cf. Medvedev, 1962) between seismic acceleration and MSK intensity, which could be estimated 8-9 at the Senerchia site, we feel the above value K = 0.15 has been exceeded at the site during the 23.11.1980 earthquake.

6 - CONCLUSION

A parametric back analysis has been used to evaluate the possible equilibrium condition of the slope.

From the numerical results the slope reached limit conditions with the arrival of the piezometric level at the surface; and this confirmed the geomorphological information that the landslide is a reactivation of an older event.

The in situ strength parameters determined by back analysis show some agreement with those obtained from the laboratory direct shear tests on the remoulded samples. The latter results may therefore represent operative strength parameters, which, however, show some differences to the residual strength, evaluated on the same remoulded samples.

This subject merits further attention. On the basis of the experimental results on undisturbed samples (Bishop et al., 1971) and remoulded samples (Maugeri, 1976), a first tentative explanation for this fact might be found in the formation of subpeaks of resistance when the sliding movement stops for a suitable length of time and the load is reduced. In this case this occurred during quieting conditions of the landslide.

The energy transmitted by the earthquake to the slope was enough to move the entire slope itself, which in a static condition had probably already been partially and independently landsliding.

The earthquake was simply a catalyst for the landslide since the slope was in a limit condition. However, it must be stressed that even if the slope had been completely drained there would have been a landslide anyway, because the seismic coefficient exceeded K = 0.15.

The above must be borne in mind when considering any future improvement in the slope stability condition. In this case also the parametric analysis (Hutchinson, 1977) could come in useful.

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ABSTRACT

Besides extinguishing many human lives, the ruinous earthquake of November 23, 1980 also destroyed or severely damaged many villages and a large part of the historic monuments (ancient cathedrals, castles, etc.) in Southern Italy. The earthquake also caused serious phenomena of ground instability: the geologic, hydrogeologic and geotechnical implications are discussed in this paper.

Ground level changes of tectonic origin, mass movements and liquefaction phenomena were very frequent. In particular, reactivation of old landslides, sometimes affecting villages, also involved areas very far away from the epicentral zone. Mass movements were possibly very intensely affected by sudden increases in pore pressure.

As a further important geological consequence of the earthquake, some springs were subjected to sudden and exceptional changes in discharge, which in certain instances were observed as early as a few weeks before the seismic event.

ABSTRACT

Le tremblement de terre catastrophique du 23 Novembre 1980 qui causa tant de victimes en Italie du Sud, fut aussi responsable de la destruction totale ou partielle d'un grand nombre de centres et de monuments historiques (anciennes cathédrales, châteaux, etc.). Le tremblement de terre fut également à l'origine des grands phénomènes d'instabilité du sol dont les corrélations géologiques, hydrogéologiques et géotechniques sont présentés dans ce rapport.

Les variations du niveau du sol d'origine tectonique, les mouvements de terrain et les phénomènes de liquéfaction ont été très fréquents. En particulier, la réactivation d'écoulements anciens qui parfois intéressaient des villages entiers, sont arrivés jusqu'à toucher des zones très éloignées du epicentre. Des augmentation soudaines de la pression interstitielle ont probablement joué un rôle essentiel dans les mouvements de terrain.

Autre conséquence importante du tremblement de terre, certaines sources d'eau ont subi des variations soudaines et exceptionnelles du débit, parfois même de semaines avant l'événement sismique.
The earthquake in question occurred at 19.34 hrs on 23 November 1980. The worst hit area is roughly elliptical in shape, measuring about 120 km by 70 km, the major axis running NW-SE. Fig. 1 illustrates the isoseismal lines and accelerations whose hypocentral parameters vis-à-vis the principal earthquake are: Latitude: 40°46'N; Longitude: 15°18'E; Depth: 18 km. It is estimated that the magnitude of the principal earthquake, which lasted over 70 seconds, was M = 6.5, while the corresponding intensity at the epicentre was around X on the Mercalli scale.

Of course, with an earthquake of such proportions, the hypocentre is a purely mathematical indication of the point of origin of the fracture, which is estimated to extend for several dozen kilometres. The speed of propagation of the fracture has been assessed at about 3 km/s.

The study of the space-time distribution of the numerous aftershocks, initially quite frequent, then gradually dying out (the maximum magnitude of these shocks was M = 5.1), has revealed the existence of four homogeneous phases of seismic activity between 23 November 1980 and 30 April 1981. Without going into minute detail, for the sake of brevity, Fig. 1 shows the epicentres of greatest magnitude during that period. These have an Apennine trend and affect an area measuring about 100 x 30 km. The greatest concentration of events is, of course, near the epicentre of the principal shocks.

The first tremor was followed about two seconds later by another of greater amplitude. This interval of time remained constant with distance. It is thus reason

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**Fig. 1** - Isoseismal lines of the earthquake of 23 November 1980 in Campania and Basilicata (Southern Italy) with indications of accelerations recorded at various monitoring stations. The epicentral map of the principal earthquake (empty circle) and the main aftershocks (full circles) up to 30 April 1981 is shown in the top right-hand corner.
able to assume that the main fracturing process involved two phases. The deductions regarding the focal mechanism, based on the sign of the first pulse of the P wave recorded at various seismograph stations, associate the earthquake with a normal fault striking roughly NW-SE, characteristic of the tension tectonics of this region. More complex activations, albeit of smaller magnitude, occurred during the aftershocks.

From the geological aspect, the physical environment of the vast area straddling the Campania-Basilicata boundary where the November 1980 earthquake occurred, is one of the most difficult of the whole of peninsular Italy (Fig. 2).

Some of the most significant points in this regard can be summarized briefly as follows:

A) Apart from the hard carbonate rocks of Monte Terminio, Monte Cervialto and Monte Marzano, most of the outcropping rocks - even outside the epicentral area proper - are characteristically of argillaceous facies. The geometric role of these rocks is thus of decisive importance as regards what recent geotechnical texts refer to as "structurally complex formations" that are particularly susceptible to macroscopic deformations or rather to mass movements in general.

B) Intense tectonic disturbances are quite evident in most of the geological formations. These have been subject to orogenic and gravitational transport on many occasions in the past and are now subject to very marked tension tectonics. The phenomena occur quite extensively as creep in some places, while in others, movement occurs instantaneously during earthquakes. Their role in the context of a more general picture of the various forms of subsidence in Italy, is evinced by the Fig. 3 diagram (Cotecchia, 1978; 1980). The geomorphology of both the Ofanto and the Sele valleys reveals the tectonic origins, even though present knowledge of the stratigraphy and tectonics is not sufficiently detailed in some cases to decide whether some of the outcropping masses are autochthonous or allochthonous. It would appear likely that the "complex formations of flysch facies" and the "multicoloured clays" were overthrust onto the Mesozoic carbonate complex in the Lower Miocene. Subsequently, they were all involved in the tectonic phases from whence may derive the graben (such as that of the Sele Valley) and the other depressions in which the softest flysch formations are now amassed, interrupted here and there by limestone and dolomite olistoliths.

C) The autochthonous sedimentary formations of the Upper Pliocene-Calabrian have also been involved in large vertical neotectonic movements, as demonstrated, for instance, by the uplift of the terraced alluvial deposits of the Ofanto, Bradano and Basento valleys.

D) The lines of demarcation between horst and graben are marked by the products of a morphological-gravitational evolution, very evidently copossed of softer, plastic rocks and by abundant cover of eluvial and detrital soils deriving from the denudation of the hillsides, widely affected by landslides and mass movements in general.

E) The area in question is affected by seismicity ranking among the most intense and frequent of the whole peninsula. Suffice it to recall that just in the territory surrounding the upper reaches of the Sele and the Ofanto there were 48 earthquakes of intensity greater than IX on the Mercalli scale between 63 AD and 1980, the rate of occurrence during the last two centuries being much higher than in the past. There was a particularly large amount of activity between 1851 and 1861 when there were two earthquakes with intensities of X, two of IX and two of VIII on the Mercalli scale.

F) The role played by nearly all the faults in the present geodynamic activity is not yet clear. However, the cause-effect relationship between tectonics and earthquakes, tectonics and landslides, and finally between tecto-
Fig. 2 - Geo-structural sketch of part of the area most directly affected by the earthquake of 23 November 1980 in Campania and Basilicata.
Fig. 3 – Tentative classification of subsidence phenomena: the noteworthy role of seismic and tectonic disturbances is evinced by the diagram.
Fig. 4 - Sketch of Senerchia landslide: The old mass movement, in the form of a flow, was reactivated by the earthquake of 23 November 1980. It grew greatly, affecting the town upstream and terminating downstream, again as a flow. The Abbazzata, Santa Lucia, Picoglia, and Acqua della Forma springs together have an average discharge of about 600 l/s. They issue forth along a fault barrier. These discharges give some idea as to the amount of ground water movement in the carbonate massif and hence of the abundant percolation in the downstream formations. 1) Jurassic limestones and dolomitic limestones; 2) Mesozoic limestone olistoliths; 3) Marly-limey-clayey flysch and multicoloured clays; 4) Cemented talus and scree; 5) Eolian products; terra rossa and black earths; pyroclastics; 6) Recent and present-day alluvium of the Sele river; 7) Landslide area reactivated by earthquake of 23 November 1980; 8) Talus cone; 9) Springs and direction of flow; 10) Main landslide scar and prevalent direction of mass movement; 11) Lateral rotational slumping; 12) Fault; 13) Section line. The section is purely hypothetical, there being no real information on depth and on the actual strike of the fault.
nics and earthquakes and landslides has been recognized for quite some time (Costecchia, 1977).

G) In this essentially mountainous and hilly region, the landform and the natural water resources — frequently aspects that aggravate the effects of earthquakes — have largely dictated the location of urban settlements, typical cases being Cassano Irpino, Andretta, Caposele, Calabritto, Queglietta, Senerchia and yet others. As a result, many towns and villages occupy sites that are particularly unfortunate from the stratigraphic, hydrogeological, geomorphological, seismic and tectonic aspects. Some of them, such as Lioni, Santomenna, Colianello and the downstream outskirts of Senerchia lie wholly or in part on talus, rock cones and alluvial fans. Not only do these deposits have inherently poor geotechnical properties, they also have groundwater movement in the immediate sub-surface (as evidenced by the large springs just upstream), and are underlain by flysch or at least essentially argillaceous ground that is frequently in the saturated state (Figs. 4, 5).

Yet other towns, such as Caposele, and Senerchia, in part, Balvano, Muro Lucano, Montella, Valva, etc. — are built on highly-fractured Mesozoic carbonate slopes, some with still-active faults. In such cases, as at Coliano, Laviano and the Queglietta Rock, the sanctuary of Senerchia and many other individual localities, the summit is often the place morphologically most exposed to earthquakes. Other towns and villages are built on the conglomeratic-sandy summits of argillaceous hills affected by slow but inexorable slope failure, such as Calitri, Risaccia, Andretta, Stigliano, Grassano, Alano, etc. (Figs. 6, 7).

H) Where the "argille varicolore scagliesi" (scaly multicoloured clays) in Tertiary flysch formations and the associated detrital cover ring the abundant

Fig. 5 — The centre of Senerchia was built and rebuilt many times on the limestones after various earthquakes in the past (those of 1733 and 1853 were disastrous). The town then expanded mainly on the vast talus cone cladding the Mesozoic limestones of the Monte Cervialto group. It was affected by an extensive landslide when the earthquake of 23 November 1980 occurred (see Geological Sketch in Fig. 4). The multiple slumping forming terracettes on the southern side of the town from the main piazza to the graveyard, and the reactivation of part of the Sele Valley Flysch beneath the cone and in tectonic contact with the Mesozoic limestones, provide tangible proof of the hazards faced by the ground "ringing" the Mesozoic limestone aquifers, as a result of the increase in pore pressure that may occur during an earthquake. a) Detail of main landslide scar which affected the main piazza and the belvedere and cut the road to Oliveto Citra, as well as the water main; on the top of the left-hand part of the photo can be seen the ruins of the old town on the limestones which were left intact by the recent earthquake; b) Saturated landslide materials (talus mixed with patches of clayey, limey, sandy flysch and multicoloured clays, whose original position is uncertain, and pyroclastic blocks) involved in the "liquefaction" phenomenon.

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carbonate aquifers (there are many such areas around the Mesozoic Picentini Mountains complex) the role played by pore pressure is particularly insidious. This pressure is generated in the infiltration beds in the ground forming the ring, downstream of the interface with the carbonate aquifers, and it increases markedly as a result of seismic vibrations.

I) In this context, account must sometimes be taken, too, of the unfavourable effect of the abrupt increases in water pressure in aquifers as a result of the vibrations, and also the changes in "storage capacity" and the permeability of aquifers following - and sometimes even preceding - an earthquake.

The area affected by the earthquake of 23 November 1980 can be divided into two sectors: west and east.

The first sector lies roughly SW of the line through Balvano, Muro Lucano, Santonenna, Caposele and Montella, and also embraces the Monti di Vietri di Potenza, Monte Marzano, Monte Cervialto and Monte Terminio. Here predominate variously tec-

Fig. 6 – Sketch of Calitri landslide, triggered off by the earthquake of 23 November 1980: from the foot of the landslide terrace, caused by broad, deep rotational slip, run two flows with a front of about 300 m in the deposition area, partly obstructing the Ofanto river. 1: Sands; 2: Probable lenticular distostromes of multicoloured clays; 3: Infra-Middle-Pliocene siltites; 4: Flow feed zone; 5: Toe of deep rotational slip; 6: Flow channels (from Del Prete and Trisorio Liuzzi, 1981).

Fig. 7 – General view of landslide which hit Calitri. The mass movements, which in times past had affected the southern slope with its Pliocene Blue Clays and interbedded Multicoloured Clays, were reactivated by the earthquake of 23 November 1980 along the lines indicated in Fig. 6.
dicate that these masses may well have moved by an appreciable though perhaps not a great amount, especially in the vertical direction. In the absence of earth tremors, these rock masses are normally reasonably stable and free from mass movements. However, with the shocks that occurred, it ensues that the risk of mass movements is quite high. Disastrous events occurred, in fact, not only in the coarse clastic Quaternary deposits (talus cones, alluvial fans, scree, etc.) resting on the rigid carbonate bedrock, but also on the steep rock slopes, especially where fractured.

The eastern sector includes the hills of the middle and upper reaches of the Sele, Ofanto, Bradano and Basento valleys. Here flysch formations predominate, mainly the Undifferentiated Sicilide Complex, Plio-Pleistocene clays and medium to coarse clastic cover materials. Because of the geomorphological, structural, drainage, hydrogeological and climatic characteristics of this sector (Apennine Basilicata and Campania) deep, widespread mass movements are very common, and so are general slope instability and delicately balanced situations on relatively steep hillsides.

From time immemorial, landslides have constituted a persistent natural calamity, threatening the existence of many towns and villages, the efficiency of the infrastructural systems and the socio-economic development of human activities.

From information acquired so far and from visual evidence, the effects of the last earthquake on slope stability and water resources can be summarized as follows:

a) As a result of the tectonic fractures that occurred, there have been marked changes in land:levels. Fig. 2 shows the checks made on some geodetic bases by the Military Geographic Institute (IGM) and the variations in levels that have occurred as a result of the earthquake, which produced sudden subsidence of up to about 80 cm. These changes in elevation are attributable to deep deformation, the cases of geotechnical compaction of previously loose surface soils as a result of the direct impact of the shock waves having been excluded. The form of the profile in Fig. 2 gives an approximate indication of several steps which may well mark the point where faults intersect the ground surface.

The extent of the destruction and of the most serious damage suffered by structures and by towns and villages as a result of the phenomena is already common knowledge, at least where the most striking and evident cases are concerned. A whole series of cracks at depth has occurred in the tunnel serving the State Electricity Board's (ENEL's) San Mango del Calore Hydroelectric Station, while linear deformations and fractures have appeared in the subsurface canal of the Apulian Water Supply (Acquedotto Pugliese), especially in the stretch joining Cassano Irpino Springs and Caposele Springs (Calore Sele tunnel) and in the Apennine tunnel beneath the Sella di Conza (Fig. 8).

Investigations are being made to ascertain if any damage has occurred in the first part of the earth dam (located in Fig. 2) under construction on the Ofanto river at Conza della Campania, as a result of the right bank being lowered 18 cm compared with the left (the crest length involved is 600 m). The spans of the long railway viaduct built to re-route the Ofantina Railway around the new reservoir, have been displaced horizontally over a considerable distance in many cases, while some of the piers are badly cracked. Many road and rail tunnels and viaducts, even a considerable distance from the epicentre area, have been found to have suffered damage in places.

b) The intensity of the earthquake was such that it initiated or - more frequently - reactivated numerous landslides and caused extensive, deep cracks to open up in the ground. Here critical equilibrium conditions - affecting naked slopes as well as slopes with towns, linear structures and engineering works of all kinds - have emerged sometimes days or even weeks after the earthquake, as the result of subsequent
meteoric events.

c) There are several instances of mass movements affecting entire hillslopes, some stretching for many kilometres and involving hundreds of millions of cubic metres of soil. In certain instances, e.g. in the Castelfranco, Calitri and Andretta areas, the displaced material slid right down to the valley bottom, restricting the channels of the rivers and streams. Figs. 6 and 7 in particular refer to the example of Calitri.

d) Local and sometimes extensive rockfalls have occurred even in the Mesozoic carbonate formations generally close to or within the epicentre area, but such occurrences are much less frequent some distance away.

e) In areas where there have been no mass movements, the earthquake has sometimes had the effect of enlarging existing tectonic joints or of opening up new fractures in rock. Good examples of this are found at Laviano, Muro Lucano and S. Fele. The phenomenon has been encountered in various geological formations and even in the debris of old landslides that are still quiescent and on alluvial plains. In unconsolidated formations, the opening up of macroscopic cracks has often been accompanied by liquefaction - a phenomenon that is dealt with ahead - with the appearance of sand volcanoes at a considerable distance from the epicentre area in some cases (Ruvo del Monte, San Giorgio La Molara, Montecalvo Irpino, Muro Lucano, Lioni, Volturara Irpina, Pontecagnano, Scafati, Buccino, etc.).

f) Even in areas far from the epicentre deep cracks frequently opened up in the ground during the earthquake. Some weeks later, when the rains came, these cracks facilitated penetration of water into the subsurface and this led to an

Fig. 8 - Some aspects of the damages produced by the earthquake of November 1980 in the Pavoncelli Tunnel (also called the Apennine Tunnel) which is part of the Apulian aqueduct. The tunnel is 10.6 km long and conveys the canal from the Sele Valley to the Ofanto Valley, crossing the Apennine Range beneath the Sella di Conza. a) Between the 4600m and the 4680 m landmarks starting from Caposele the inverted arch in brick masonry shows crushing and cusp lifting. The old rails used for maintenance show severe deformation. At this site, the depth of the top cover is 350 m. This type of failure also involves an altimetrical deformation of the level which is now being investigated; b) A shear failure with prevalingly horizontal sliding of the stone revetment at the 8890 m landmark marking the transition between the multicoloured scaly clays (upstream side) and the Pliocene blue clays of the Ofanto basin.
unusual rise in pore pressure, which caused fairly extensive multiple slumping and other types of mass movement. So many areas that were apparently spared by the earthquake are still living under the threat of possible mass movements in course of development (Stigliano, Grassano, Aliano, Ferrandina, etc.).

g) The phenomena encountered in the Caposele and Cassano Irpino springs area following the earthquake of 23 November 1980 provide a good example of the complexity and multiplicity of the interrelations among hydrogeological conditions, seismic events, modifications in elevation and slope movements. The most noteworthy aspect in this respect concerns the variations in quality and changes in discharges of the springs (Fig. 9). The hydrograph for the Sanità di Caposele Spring shows that it was at a low-discharge stage on 23 November 1980. Then from 25 November 1980 to 5 January 1981 the discharge rose rapidly and continuously from 4210 l/s to 7270 l/s. The daily rate of increase following the earthquake reached 400 l/s, a value never before recorded. The hydrogeological studies made in this regard (Cotecchia and Salvemini, 1981) and the correlations between present-day and historical data on the spring, clearly show that the earthquake was responsible for the fact that the discharge rose to a record 7270 l/s. This level was attained in forty days, starting from a presumed low-flow condition, the maximum value being recorded in December. It is only possible to speculate as to the causes of this exceptional change in discharge and the concomitant modifications in the subsurface hydrological regime. The answer probably lies in a relative lowering of the spring threshold, since Caposele must be considered a barrier spring and the ground forming the impermeable plug in the carbonate aquifer is chronically subject to mass movements here. It has been ascertained in the neighbourhood that seismic subsidence along a number of lines in the zone, caused differential horizontal and vertical variations in position, particularly along the structural trends. In the case of Caposele, the size and geometric configuration of the aquifer are such that the impermeable silt of the headworks yard immediately against the limestone slopes would only have to be lowered a dozen or so centimetres to explain the discharge-increase situation.

Fig. 9 - Discharge hydrographs of the Cassano Irpino Group of Springs (Bagno, Pollentina, Peschiera and Prete) and of the Sanità di Caposele Spring before and after the earthquake in Campania and Basilicata on 23 November 1980. Comparison of these hydrographs with those for the average year reveals the big increase in discharge as a result of the quake.

Turning now to the discharge variations that occurred in the Cassano Irpino group of springs, records show that in recent years the regime of the four springs (Peschiera, Bagno, Prete and Pollentina) has been distinguished by a low-discharge period that lasted virtually for the whole of November and often into the following months too. Indeed, the rise in the flow rate never started before December and sometimes not even before February. Once the rise in discharge started, it generally took three or four months to attain the peak value. Unlike the case of the Caposele
Spring, when the earthquake of 23 November 1980 occurred the discharge values of the four springs had already started their upward climb a couple of weeks earlier, the increase in flow rate being exceptionally steep, as at Capeosele. The total discharge of the Cassano Irpino group of springs may be considered, for the sake of simplicity. As stated, this started about a fortnight prior to the earthquake, during which time the discharge rose from 2900 to 4060 l/s at a daily rate of between 80 and 100 l/s. After a brief pause in the upward trend in the days preceding the earthquake, the discharge started to rise again even more rapidly, reaching an all-time high of over 5000 l/s in early December.

Investigations on changes in spring regimes in the earthquake area thus provide many important pointers for a better understanding of the multiple relationships interlinking earthquakes, slope stability and hydrogeological behaviour of deep aquifers.

Two National Research Council (CNR) projects "Geodynamics" and "Mass Movement Phenomena" were launched immediately after the earthquake to investigate in depth the matters outlined above. The projects are designed to shed more light on the static and dynamic aspects of the territory affected by the earthquake, not only within the geo-structural context of the environment in which they occur, but also as regards the geomechanical behaviour of the soils in the various cases involved.

The most recent published material on earthquakes in various parts of the world (Alaska, California, Japan, Chile, Romania, etc.) provides an important point of reference for broaching the by no means easy problem of evaluating stability. As is generally known, much intensive research has been performed in recent years on the dynamic behaviour of soils, especially in relation to problems connected with the safety of high-risk plants.

Two catastrophic events, both of which occurred in 1964, provide a broad basis for observation and experimentation to acquire knowledge on the subject, namely the Alaska earthquake and the Niigata earthquake in Japan. The need to resolve problems concerning structures requiring a large in-built margin of safety, such as nuclear power stations, offshore oil platforms, etc. provided the initial spur to acquire relevant geotechnical information.

In Niigata province, textbook examples were observed of the liquefaction of loose sandy deposits. Sand gushed forth like a spring and large areas of quicksand developed. The damage in Alaska was just as great, though there the deposits involved were mainly clayey (e.g. the Turnagain landslide at Anchorage).

Italy's seismic history also provides outstanding examples of these phenomena. Observations and descriptions are available at least for earthquakes that occurred during the last two centuries, namely from that in Calabria in 1783 to the recent one in Friuli in 1976.

It is also worth recalling here that the 1977 Romanian earthquake which had its epicentre in the Vrancea Mountains, produced liquefaction on the banks of the Danube about 250 km away.

Numerous examples of liquefaction were recorded during the Campania and Basilicata earthquakes, as indicated in this paper. Some of these occurred in areas up to 70 or 80 km from the epicentre, for instance, in several localities along the Tyrrenian coast, where there are clastic and pyroclastic deposits containing ground waters (Scafati). Such phenomena may well have played a decisive role, too, along the slip planes of many slumps, and this is an aspect that warrants more attention in future studies.

Urban and industrial structures and linear infrastructure systems such as roads, canals, etc., are all particularly vulnerable to liquefaction. This matter, of course, assumes special importance when tackling the pressing problems of reconstruction of the earthquake-stricken areas and when framing the necessary preventive studies.

Studies designed to examine soil stability during earthquakes certainly have wide-ranging, complex implications, involving as they do such disciplines as geo-
logy, geophysics, seismology and geotechnics. Though each of these disciplines today certainly contributes in a valid manner to resolving the central problem, such contributions tend to be on an individual basis and therefore lack the force and feedback there would be if they were coordinated. What those responsible for the reconstruction of the earthquake-stricken areas need is the best possible pooling of all contributions for tackling the detailed problems involved and for predicting the likelihood of the occurrence of given phenomena. Progress has been made in this direction in recent years but much still remains to be done.

The methods of seismic analysis still generally ignore the role of the initial tension state, when assessing the possibility of instantaneous or gradual slope failure. It often happens that local geological characteristics do not receive sufficient attention when performing geo-mechanical analysis, yet detailed variability here can be of decisive influence as regards the destructive effects of an earthquake. This fact is well borne out by the census of damage suffered by structurally identical buildings set on different soils, albeit belonging to the same geological "unit".

Another basic essential for ascertaining the effects of earthquakes on soils, or rather, on the strength of soils when subject to cyclic stress, is the in-situ determination of the variation of pore pressure during tremors. Provision should be made for a network of automated piezometers in areas with a seismic hazard. Such piezometers could be installed in the most significant zones and the network should be more closely knit than that used for monitoring acceleration.

To ensure that the network is set up in the best locations, it is essential that a check be made on the behaviour of soils and structures during the earthquake, to be carried out in the post-seismic period. In the absence of instruments for making checks at the moment of the shock (benchmarks, inclinometers, etc.) this post-seismic investigation of behaviour can provide useful information on the non-macroscopic shifts of ground that occurred and which were responsible for reactivating old landslides and for moving overthrust blocks about which exhaustive information is not always available in the area affected by the Campania and Basilicata earthquake.

In a broader view, "seismic zoning" assumes much wider significance than the mere characterization of the overall seismic hazard of an area, even though understood as the superimposition of various factors that go to define this. Adequate seismic zoning stems from the superimposition of a number of specific studies and a multiplicity of thematic maps, supported by systematic checks. The seismic hazard of an area, in fact, depends on at least three factors:

a) The presence of structures generating shocks in or near the area.

b) The vertical and horizontal structural attitude of the various rock masses in the zoning area (structural and stratigraphic geological model) and the final geomorphological configuration.

c) The mechanical properties of the ground within which the seismic stresses are propagated, especially unit weight, modulus of elasticity, modulus of shear, Poisson's ratio, and shear strength parameters.

A complementary, though indispensable, aspect as regards seismic zoning concerns the preparation of thematic maps of the seismic hazard plotted from historical analysis of earthquakes: these define the hazard on a probability basis by reference to shocks of predetermined intensity and statistically computed return periods.

Taken altogether, these elements help define the "seismic response" of a site. In other words, the lack of homogeneity and discontinuity of the bedrock, the geomorphological and structural factors, and the geotechnical characteristics of the cover soils considered together permit the delineation of areas which behave differently when subjected to the same shock. The isoseismal lines, therefore, do not run out concentrically but take the form of elongated ellipsoids, often of irregular configuration even quite close to the epicentre area.

The two CNR projects for the earth-
quake-stricken area described are framed along these lines.

In Campania and Basilicata the complex deformational picture produced by the earthquake has once again revealed just how much has to be done to arrive at a correct interpretation of geomorphological configurations.

For example, very little is known about the profound geomorphological evolutions produced by past earthquakes, while more thorough investigation is needed of the modifications in landform caused by the last changes in shorelines, which have certainly had an effect on the deepening of drainage networks, on erosion processes and on slope movements, too. In particular, as regards the large area hit by the recent earthquake, the geomorphological modifications that occurred in the Quaternary following the eruption of the Vulture volcano have to be carefully evaluated, within the context of geological and geotechnical conditions that now control the stability of some slopes particularly sensitive to seismic shock. The earthquake of 23 November 1980 clearly showed that were there exists overthrusting and the lithological composition is fragile, the superimposition of seismic shock on areas in rapid geomorphological evolution causes a big increase in the geological risk.

It is in this light that all those basic studies capable of ensuring the univocal definition of the intrinsic physical aspects of the different parts of the territory must be pursued coherently as a matter of priority. In this manner it will be possible to acquire a "knowledge bank" and "pilot examples" on which the "zoning" of the earthquake-stricken areas for reconstruction must necessarily be based.

ACKNOWLEDGEMENTS

The author is grateful to Alfredo Calvaruso and Leopoldo Romanazzi for their contribution to this paper.
SEISMO-TECTONIC ZONING AND SEISMIC RISK ANALYSES OF A PROJECT SITE

LA ZONATION SEISMO-TECTONIQUE ET L'ANALYSE DE RISQUE SEISMIQUE D'UN SITE DE PROJET

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ABSTRACT
The earthquakes in the seismic province are distributed mainly along the NNE tectonic line in the Neocathaysian system. The province is divided into four seismo-tectonic zones in accordance with the homogeneity of geological tectonic zones and the similarity of seismic activities. Fourteen active faults are identified in this province with a radius of 250 km from the site, and the maximum probable earthquakes have been evaluated. A statistic analysis on the relation of magnitude to frequency for each seismo-tectonic zone is completed. The attenuation of peak ground acceleration with distance has been studied, and the parameters of ground motion has also been estimated. Finally, the curves of the relationship for exceedence probability with acceleration are given for the aseismatic design.

Les séismes dans la région séismique sont distribués principalement selon la NNE ligne tectonique dans le système Néocathaysien. La région est divisée en quatre zones seismo-tectoniques en fonction des zones géologiques tectoniques et des activités similaires des séismes. Quatorze faux lits actifs sont identifiés dans la région avec un rayon de 250 km de la site et les séismes les plus grands possibles ont été évalués. Une analyse statistique sur la relation entre magnitude et fréquence pour chaque zone seismo-tectonique est complétée. L'atténuation d'accélération de sommet de terre avec distance a été étudiée, et le paramètre du mouvement de la terre a aussi été estimé. Finalement, les courbes de la relation pour la probabilité excédante avec l'accélération sont données pour le dessin aseismatique.

1. INTRODUCTION
The site studied is located northeast of the city of Tangshan, Hopei Province, China about 50 km from the epicenter of earthquake (R 7.8) occurred on July 28th, 1976. This study aims to evaluate the probable seismic risk of the engineering facilities at the project site within its lifetime, and to derive the ground motion parameters needed for the aseismatic design.

Prior to the study, the authors have
collected and looked up a considerable number of information concerning regional geology, seismotectonics, historical seismicity and related geophysical measurements. On this basis, the tectonic setting and the fundamental features of seismogeology of this region have been studied; the seismic activity associated with the historical seismicity have also been studied; and the earthquake peak ground acceleration attenuation with distance under the site condition has been estimated; finally, the seismic risk analysis and relevant engineering assessment have been made by probabilistic method.

2. REGIONAL TECTONIC SETTING

The basic tectonic setup of the North China geologic province consists of large-scale Neocathaysian NNE trending tectonic system and complicated transversal tectonic zone with EW strike (see Fig. 1).

![Fig. 1](image-url)

The available data suggest that the dominant tectonic system with evident activities is seen to strike in a NNE direction. The EW tectonic faults pertain to relatively paleotectonic system, but exhibiting intensive activity along the southern edge. As to the relation on these two large tectonic systems, the EW striking tectonic system is generally traversed by the NNE trending ones.

In accordance with the earthquake catalog, in the eastern part of China, a series of relatively intensive earthquake successively occurred and distributed along
the Neoathysian fault system in the past ten years and more. Accordingly, it can be considered that the predominant compressional stress of this province trends north-north-eastward and north-eastward. The earthquakes occurring in the province are mainly along NNE strike faults due to the effect of compressional stresses. The intensive earthquakes frequently occur at the composite parts of these active faults and others with different strike (SW or NW). However, NNE trending active fault of Neoathysian system still plays a predominant role.

3. REGIONAL SEISMIC FEATURES AND SEISMO-TEC TONIC ZONATION

The eastern part of the North China Province and the littoral area pertain tectonically to the second settling zone of Neoathysian system — Hopei plain trough zone. As the fault activities and the differential movement along the faults have been intensive since the Neozoic, especially the Quaternary period, this zone has formed a seismically active zone controlled by the settling zone. This zone is a part of the North China seismic Province. Available data concerning the seismic activities suggest that the distribution of earthquakes is evidently consistent with that of the active fault zone, particularly the tectonics of Neoathysian system within the settling zone, thus the dominant NNE trending seismically active zone has formed. Because of the earthquakes controlled by the unified tectonic stress field of the North China Province within this zone occur with sliding along the existing fault plane, so all of them pertain to shallow focus shocks at depths from 15 to 30 km.

According to the homogeneity of seismotectonic feature and the similarity of the seismic activity, the entire settling zone can be divided into four NE striking seismically active tectonic zones, i.e., (1) Chang Shen Upheaval zone (Z1); (2) settling zone in the middle part of Hopei (Z2); (3) Taihang Shan piedmont fault zone (Z3); (4) Tanchen-Lujiang fault zone (Z4).

In order to study the earthquake ground motion at the site, 14 active faults located within 250 km of the site have been identified (see Tab. 1 and Fig. 1).

<table>
<thead>
<tr>
<th>Fault Number</th>
<th>Fault Name</th>
<th>Strike</th>
<th>Fault Length (km)</th>
<th>MPE (M)</th>
<th>Pertaining to seismo-tectonic zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-1</td>
<td>NNE</td>
<td>107</td>
<td>7.1</td>
<td>Z1</td>
</tr>
<tr>
<td>2</td>
<td>F-2</td>
<td>&quot;</td>
<td>138</td>
<td>7.2</td>
<td>&quot;</td>
</tr>
<tr>
<td>3</td>
<td>F-3</td>
<td>EW</td>
<td>53</td>
<td>6.8</td>
<td>&quot;</td>
</tr>
<tr>
<td>4</td>
<td>F-4</td>
<td>NE</td>
<td>107</td>
<td>7.1</td>
<td>&quot;</td>
</tr>
<tr>
<td>5</td>
<td>F-5</td>
<td>NNE</td>
<td>410</td>
<td>7.8</td>
<td>&quot;</td>
</tr>
<tr>
<td>6</td>
<td>F-6</td>
<td>&quot;</td>
<td>157</td>
<td>7.3</td>
<td>&quot;</td>
</tr>
<tr>
<td>7</td>
<td>F-7</td>
<td>&quot;</td>
<td>105</td>
<td>7.1</td>
<td>Z2</td>
</tr>
<tr>
<td>8</td>
<td>F-8</td>
<td>EW</td>
<td>85</td>
<td>7.0</td>
<td>&quot;</td>
</tr>
<tr>
<td>9</td>
<td>F-9</td>
<td>NNE</td>
<td>123</td>
<td>7.1</td>
<td>&quot;</td>
</tr>
<tr>
<td>10</td>
<td>F-10</td>
<td>&quot;</td>
<td>151</td>
<td>7.2</td>
<td>&quot;</td>
</tr>
<tr>
<td>11</td>
<td>F-11</td>
<td>&quot;</td>
<td>145</td>
<td>7.2</td>
<td>Z3</td>
</tr>
<tr>
<td>12</td>
<td>F-12</td>
<td>&quot;</td>
<td>300</td>
<td>7.6</td>
<td>&quot;</td>
</tr>
<tr>
<td>13</td>
<td>F-13</td>
<td>EW</td>
<td>170</td>
<td>7.3</td>
<td>&quot;</td>
</tr>
<tr>
<td>14</td>
<td>F-14</td>
<td>NNE</td>
<td>3000</td>
<td>8.5</td>
<td>Z4</td>
</tr>
</tbody>
</table>

4. EVALUATION OF GROUND MOTION

(1) Maximum Probable Earthquake (MPE)

The Maximum Probable Earthquake (MPE) is defined as the largest earthquake which can be postulated for a fault solely on
seismo-tectonic consideration in the future. In this study, it is determined by the comprehensive analysis of the results evaluated from the empirical equation for estimating the magnitude and the catalog of historical seismicity.

The MFZ for every single fault are summarized in Tab.1.

The earthquake magnitude obtained from the empirical equation is generally a function of the assumed length of rupture zone. For the evaluation of the MFZ event, 35 percent to 50 percent fault rupture criteria were assumed. On the basis of comparison and analysis, the equation proposed by Patwardhan et al. (1976), as modified by Slemmons (1977) has been adopted. This magnitude-length correlations are given by Equations (1) and (2), corresponding to \( M > 6.4 \) and \( M < 6.4 \) respectively.

\[
\begin{align*}
N &= 1.054 \log L + 5.26 \quad (1) \\
N &= 2.65 \log L + 3.15 \quad (2)
\end{align*}
\]

Where \( L \) represents the length of the assumed rupture zone (in this calculation, \( L \) is considered as 50 percent of the entire fault length).

2. Acceleration attenuation

The relationships for the maximum ground acceleration with magnitude and distance are usually proposed under specific geological conditions and focal depth. In order to give a rational evaluation of ground motion attenuation of the site of study, associated with the site geological conditions, 10 attenuation modes from McGuire (1974), Esteva (1969) and others have been taken for calculation and comparison. Finally the McGuire mode (1974) was adopted for calculating the peak ground acceleration attenuation of 14 active faults. The relationship is given as follows:

\[
a_{\text{max}} = \frac{(472) \times 10^{0.278M}}{(R + 25)^{1.3}} \quad (3)
\]

Where,

\( R \) - hypocentral distance (km)

\( M \) - earthquake magnitude (Richter)

\( a_{\text{max}} \) - maximum ground acceleration (cm/sec^2)

it is noted the standard deviation of \( a_{\text{max}} \) is \( \sigma_{\log a_{\text{max}}} = 0.222 \).

5. Magnitude-Frequency Relationship

The seismic activity of North China exhibits a clear periodicity of some 300 years. Taking into account of this distinguishing characteristics, the historical seismicity counted is selected from the earthquake catalog from 1369 to 1976, i.e., earthquakes of two complete period of seismic activity. In addition, as the actual difference of seismic activity for each active tectonic zone is considered, and any one of them is being controlled by the same deep fault tectonics, accordingly, the frequency statistics was completed for each of the tectonic zones.

Within the period of activity for any active fault zone, the natural logarithm of events of a given magnitude occurring in a region is assumed to be linearly proportional to the magnitude, i.e.

\[
\ln N (M) = \alpha + \beta M \quad (4)
\]

where,

\( N (M) \) - number of events for magnitude \( M \)

\( \alpha, \beta \) - constants

The relationship for \( \ln N (M) \) with Magnitude shown in Fig.2 demonstrates the statistical results of four active seismo-tectonic zones in the province of study. 73 earthquake events of \( M \geq 4.0 \) are included in the statistics.
For the normalization of the statistic data, as per reference (5), assuming
\[ N'(m) = \frac{N(m)}{LT} \]  
(5)

where,
- \( L \) - length of the active tectonic zone (km)
- \( T \) - time for which the data was obtained (year)

\( N'(m) \) - normalized mean number of events above magnitude \( m \) for unit time (1 year) and unit length

Then,
\[ \ln N'(m) = a' + b_1 \ln N \]

where,
\[ a' = a - \ln(LT) \]

The values of \( a' \), \( \beta \) for each seismo-tectonic zone are presented in Tab. 2.

<table>
<thead>
<tr>
<th>Name of seismo-tectonic zone</th>
<th>Value of ( a' )</th>
<th>Value of ( \beta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Z_1 )</td>
<td>-4.9547</td>
<td>-0.9152</td>
</tr>
<tr>
<td>( Z_2 )</td>
<td>-5.2489</td>
<td>-0.7436</td>
</tr>
<tr>
<td>( Z_3 )</td>
<td>-3.7459</td>
<td>-0.9528</td>
</tr>
<tr>
<td>( Z_4 )</td>
<td>-5.7917</td>
<td>-0.8743</td>
</tr>
</tbody>
</table>

6. PROBABILISTIC ANALYSIS OF SEISMIC RISK
AND ENGINEERING ASSESSMENT.

To study the probable seismic risk of the site for a given time, a probabilistic method is applied for the evaluation of the peak ground acceleration that could be occurred in the future.

(i) Cumulative distribution function of the peak ground acceleration

On the basis of the characteristics of earthquake activities, the occurrence rate of earthquake with time can be modelled as Poisson arrivals. The assumptions are: earthquakes are temporally and spatially independent, and the probability that two seismic events will take place at the same place and at the same instant of time approaches zero. Thus, within the economic life of a given facility for the line source, the probability of at least one event of magnitude above \( m \) in time period \( t \) is given as follows:

\[ P(N \geq m, t) = 1 - \exp \left(- \int_1^2 N'(m) \, dL \, t \right) \]

(7)

Let the acceleration
\[ a = \frac{b_1 \exp (b_2 m)}{(R + b_4)^{b_3}} \]

then the probabilistic distribution of maximum ground motion is expressed as:

\[ F(A \geq a, t) = P \left( \frac{b_1 \exp (b_2 m)}{(R + b_4)^{b_3}} \geq a \right) \]

(8)

= \int_1^2 \left( \frac{b_1 \exp (b_2 m)}{(R + b_4)^{b_3}} \right) \, dL \]  
(9)

where,
\[ R_k = \text{hypocentral distance (km)}; \]
\[ l_1, l_2 = \text{integrated length of line source (km)}; \]
\[ i = \text{number of seismic zone (i = 1, 2, \ldots \ldots, n)}; \]
\[ j = \text{number of line source (j = 1, 2, \ldots \ldots, n)}; \]
\[ k = \text{number of small segments for every line source (k = 1, 2, \ldots \ldots, h)}; \]
\[ b_1 = 472; b_2 = 0.64; b_3 = 1.3 \text{ and } b_4 = 25, \]

when the McGuire attenuation relationship is employed.

Allowing consideration of 50 years of exposure time, the probabilistic cumulative distribution function of maximum ground acceleration at the site studied.
has been evaluated from Equation (9) as shown in Fig. 3. In this analysis, as the characteristics of earthquake activity have been taken into account, the values of \( \alpha \) and \( \beta \) for each seismo-tectonic zone are assumed as constants. It should be pointed out, having consulted reference (5), we adopted the above-mentioned equations for this evaluation.

(2) Return period

The return period is defined as the recurrence interval of a given earthquake event.

Equation (9) suggests the probability of at least one event of the peak ground acceleration \( A \) exceeding a specific value \( a \) (i.e., \( P_t(A \geq a) \)) in time period \( t \) at the site, accordingly, the probability of event smaller than \( a \) in the same time period can be expressed as:

\[
P_t(0) = 1 - P_t(A \geq a)
\]

From the binomial probability law,

\[
P_t(0) = (\begin{pmatrix} t \\ 0 \end{pmatrix}) P^0 (1-P)^t
\]

\[
= (1-P)^t
\]

(11)

where \( P \) is the mean annual risk of exceedence, which indicates the average probability of events occurred annually.

The return period is:

\[
T = \frac{1}{P}
\]

(12)

The calculated relationship of maximum ground acceleration and probability of return period at the site are shown in Fig. 4.

(3) Engineering assessment

In the preceding paragraph, the relationship for the peak ground acceleration of a site of study with corresponding return period has been studied (see Fig. 4), which indicates for a specific return period, there is a corresponding peak ground acceleration.

In order to aid the designer to select the ground motion parameters corresponding to acceptable risk level, the relationship between the lifetime of facilities, acceptable risk level and return period has been studied.

Again, assuming the earthquake events follow the Poisson distribution law in time, then during the lifetime of a facility, the probability exceeding a specific acceleration can be expressed as:

\[
P(A_1 \geq a) = 1 - \exp \left(-\frac{t}{T_1}\right)
\]

(13)

where,

\( t \) - the lifetime of a facility

\( T_1 \) - the return period of the earthquake acceleration

The results obtained from Equation (13) are shown in Fig. 5.
For selecting a rational acceptable level of risk, the significance, class, size and economic life of buildings or structures, as well as the possible consequences of failure should always be considered. In accordance with the factors considered the acceptable risk level of the site studied of 20 percent has been adopted. On the basis of this level, assuming a 50 year life of structure, $a_{\text{max}} = 0.20g$ can be obtained from Fig. 5, and for a 30 year life of structure, $a_{\text{max}} = 0.17g$ under the same risk level.

Final results of seismic risk analysis in this study are presented in Fig. 5.

**REFERENCE**


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SEISMIC WAVE FIELD AND ITS DAMAGING EFFECT

CHAMP D’ONDE SISMIQUE ET SON EFFET NUISIBLE

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SYNOPSIS
This paper presents from engineering geological point of view a new concept, i.e. many of the surface damages are not caused by seismic force but only the results of particular mode and behavior of surface wavefield. Various surface damages encountered in Tangshan strong earthquake zones have been given in this paper as real case histories which clearly illustrate that the engineering geological conditions of site or terrain will designate certain particular behavior of surface wavefield, and further, will specify the damaging manner of ground. Based on this analysis, the author suggested necessary measures to prevent the occurrence of such damages and methods of making earthquake engineering geological assessment. This would supplement each other with the seismic force theory to make the seismic design of a project more effective and reasonable.

RESUME
Ce papier presente une idee neuve au point de vue de geologie de l’ingénieur sismique, i.e. beaucoup de resultent du comportement et de la forme particuliére des mouvements de champ d’onde au lieu de la force sismique. Les exemples numerus des differents dommages de terre superficialie dans la zone de secousse sismique de la ville de Tangshan prouvent que ce sont les condition de l’ingénierie geologique de site qui déterminent les caracteristiques de mouvement en place, puis les formes de dommage. L’auteur propose les mesures pour eviter les dommages sur le site et les methodes d’évaluation de geologie de l’ingénieur sismique. Avec la théorie de force sismique, cette idee fait plus efficaces les mesures parasismiques de l’ingénier.

1. Concept of Seismic Wave Field
On the traditional viewpoint of earthquake engineering, damages of ground facilities are caused by seismic forces. However, investigation in earthquake zones proved that many of the ground damages were closely related to the shaking or waving behavior of a site covering a certain area which implies relative movement encountered rather than simply of a certain point. Therefore, the "wave field" defined here denotes the ground movement as a whole which covers an area much larger than that overlain by the foundation of a structure with limited size.

In this paper, some case histories of ground damages are presented showing particular evidences as follows:
(1) The damaging manner of ground may reflect the real features of wave field actually recorded as a whole of ground movement at a site during main shock.
(2) Damages of extremely light structures on ground may not be due to seismic load, because their mass is too small to exert seismic force large enough to be destructive.
(3) Most of the strong movements of ground are plastic and irreversible rather
The common characters of the all bridges destroyed seismically are; (1) Concrete plates of the bridge damaged intermittently and fell down only in longitudinal direction even though much smaller rigidity exists in latitudinal direction; (2) damaged portions were in successive and progressive failure, i.e. damaging effect reoccurred in successive events; (3) central part of the bridge damaged more seriously; (4) all the plates fell down in same direction.

The above mentioned phenomena exhibited very unusual manner and can hardly be explained by seismic force theory due to the facts: (1) The mass of the ground structure is too small to produce seismic force acting on the structure even the acceleration due to earthquake may be large enough; (2) That bridge destroyed intermittently cannot be imagined as the input accelerations on the whole bridge distributed in the same manner.

Intermittent or rhythmically distributed damages often occur in the central part of river bends and oriented in the axial direction. Plate 2.1.2 show the actually damaged site of a hospital in Tengshan earthquake. The stripes in white are the macroscopic features of soil liquefaction. There are equal spacings between stripes which were actually the tensile cracks of sand boiling or series of sand boils. The most unusual phenomena were intermittent damages of five single storey buildings which collapsed and stood still in every other line. By the author's opinion, this was formed due to stationary waves took place in the middle part between two parallel borders which reflected the travelling surface wave. By the definition of stationary waves, the general form of vibration can be written as

\[ u(x,t) = X(x)T(t) \]

By discrete variable method, we have

\[ X(x) = C \sin \frac{\pi x}{L} \]

\[ T(t) = A \sin \frac{\pi t}{T} + B \cos \frac{\pi t}{T} \]

where the amplitude \( X \) at position \( x \) is \( X(x) \), and the phasic state \( T \) is expressed in terms of time \( t \) i.e. \( T(t) \). 

\( A, B \) are constants of integration. Thus, for stationary wave, we have
For each \( n \) value, there is a specific stationary wave.

From equation (2.1.2) we may take as the distance between two sides of a certain segment of the parabolic shaped river bend. "a" can be considered as half wave length \( (a = \frac{2\pi L}{n}) \). It is obvious that under a stationary wave action, \( u \) is apt to become maximum when \( \frac{L}{a} \) ratio is neither too high, nor too low. That means such intermittent or rhythmical damage is most likely to occur in a river bend of moderate size. By our experience, \( \frac{L}{a} \) ratio would be 10-30 for such case.

The interference of waves may take place in many cases and may exhibit in various manner. In a bound site such as lake, valley, river bed and even a pool, the wave action may be excited due to local effects of topography and geomorphology.

Knowing the surface wave length of the riverbed, we may estimate, even roughly, the possible damaging positions of the bridge in order to take remedial measures.

Plate 2.2.1 Damage due to strong vertical movement

2.2 Strong vertical movement

Plate 2.2.1 is one of many actual damage due to strong vertical movement in Tangshan earthquake. Evidence shows that vertical seismic force was so strong that the brick chimney stack was broken into 8 segments in its middle part and meanwhile lost its upper third. Very unusual features can be seen, i.e. (1) Chimney stack was broken into multiple segments simultaneously; (2) Rupture on each segment in middle part demonstrates a vertical compressive failure pattern; (3) No evidence shows horizontal seismic force did ever act on the broken segments.

Another evidence of vertical seismic movement is shown in Plate 2.2.2 in which a railroad and subgrade underneath waved and deformed as it looks to be. This illustrates the ground movement on that site was mainly vertical and remains its original manner seemingly to be a sequence of surficial wave, say Raleigh wave action. However, the wave length of the rail is smaller than that of Raleigh wave. This might be due to the intersection of the natural ground and the subgrade together with the rail.

In the latter case, the mass of the structures on ground were not large enough to produce destructive lateral seismic force, so the only factor might be the wavy deformation of ground associated with the subgrade which exceeds the elastic limit of deformation of themselves.

2.3 Torsional wavefield

When shear wave propagates along ground surface, the Love wave is likely to occur which may cause much stronger torsional movement of ground due to unequal phased displacement horizontally.

Plate 2.3.1 is the damaged railroad due to horizontal distortion. Such a deformational pattern clearly recorded the particular ground movement during main shaking phase i.e. mainly horizontally polarized SH wave. It can be seen that such deformation only occur in a limited area which covers several wave length.
theory, it can be proved that so long as there is an inclined interface above or below the liquefied layer, there will be a rotational field (Wang 1961).

Plate 2.3.2 A 4-storey building broke down due to SH wave

Torsional wave field can also be found in liquefied site surrounded by river bends. Plate 2.3.3. shows an overview feature of sandbells which is so called the "vortex pattern" of macroscopic liquefaction. By the analysis of field...
Plate 2.4.2. gives another example of
sine kind, where a highway bridge was
wholly destroyed during earthquake in the
manner of strong swaying and caused
the concrete slabs to fly over and overlapped
one after another. However, the single
storey warehouses were standing still.

Plate 2.4.2
Bridge and single storey house
behaved quite differently due to
resonance effect.

From actual data, the proper period
of the soft ground in Ninghe county,
Tienjing ranges about 0.6-0.8 sec., which
closely approaches to the period of the
big R.C. water tank with brick bearing
wall and that of the precasted R.C. bri-
dge. On the contrary, it is far from
the period of single story building
which is nearly to the range of 0.2-0.3
sec. That’s why they were saved from
that overwhelming seismic disaster.

2.5. Over-wavelength movement
It can be well understood that
during an earthquake the structure on
ground will behave together with ground
soil as a whole. Never-the-less, if the
dynamic behavior or the geometric para-
ters of them are quite different, there
will be unconfomerable movement between the
two. Then damages will take place where
unconformity exists. The more energy
loss the structure exhausted in unconfor-
mable movement, the more serious damage
the structure will undergo.

Referring to Yamahara’s work (1967),
suppose the ground shaking is propagated
in a manner of continuously harmonic
waving, thus we can derive the relation
between the longitudinal component of
ground movement and the vibration of the
structure, Assuming to be the peak
amplitude of ground vibration, then the
seismic displacement \( u \) at certain point
and certain instant \( t \) is

\[
U = u_0 \cos \frac{2\pi}{T} (t - \frac{L}{T} x) \quad (2.5.1)
\]

where

\[
T \quad \text{--- period of ground vibration}
\]

\[
L \quad \text{--- wave length of ground vibration}
\]

For calculating the vibrational displace-
ment of the structure which is assumed to
be rigid, and we can deal with the average
displacement \( \bar{U} \) instead of \( u \). Thus we
have

\[
U = \bar{U} = \int_0^L u_0 \cos \frac{2\pi}{T} (t - \frac{L}{T} x) dx \quad (2.5.2)
\]

The max. displacement will take place as
\( t = 0 \)

\[
\therefore \quad U_{max} = \int_0^L u_0 \cos \frac{2\pi}{T} x dx
\]

\[
= u_0 \frac{T}{2\pi} \sin \frac{\pi}{T} \quad (2.5.3)
\]

and

\[
U_{2\pi} = |U_{2\pi}| = 0
\]

Let \( r_0 = \frac{U_{max}}{u_0} \), \( L_0 = \frac{L}{T} \)

then

\[
r_0 = \frac{1}{T} \sin \frac{\pi}{r_0}
\]

as \( L \gg \lambda \), \( r_0 \rightarrow 0 \)

\[
\therefore \quad \lim r_0 = \frac{h}{h_0} \quad \lim \frac{h}{h_0} \sin \frac{\pi}{r_0} = 1
\]

as \( L \ll \lambda \), \( r_0 \rightarrow \infty \), \( r_0 \rightarrow 0 \)

\[
\therefore \quad \lim r_0 = h_0 \quad \lim \frac{h}{h_0} \sin \frac{\pi}{r_0} = 0
\]

\[
L \rightarrow 0 \quad r_0 \rightarrow 0
\]

Value of \( U_{max} \) versus \( r_0 \) can be plot-
ted as shown in Fig. 2.5.1.
It is obvious, when \( L \gg \lambda \), i.e. whenever the length of the structure is smaller than the wave length of the ground no harmful ground movement to the structure in the sense of unconfomable vibration will occur. In order to avoid the above phenomena it is necessary to restrict the length of the structure within \( L \) as a critical limit, otherwise necessary flexible joint or separating measures are hopefully provided.

3. Foundational analysis of wave field

The above mentioned case histories provide many backgrounds illustrating damages on ground are often the results of wave field effect on a site. Any point within such field will behave a cyclic motion about its centered position of equilibrium. So the ground in vibration will deform and absorb the kinetic energy of vibration which will be in turn transferred into potential energy. Therefore, wave motion itself is a transferring procedure of energy.

Let \( dV \) be the volume of an element in an elastic half space, \( \rho \) is its density. As a wave front with a velocity \( V_s \) is passing by, the element will gain a kinetic energy

\[
dW_k = \frac{1}{2} \rho dV V_s^2 \quad \text{(3.1.1)}
\]

The potential energy gained during vibration is equal to the kinetic energy provided damping can be neglected. Namely,

\[
dW_p = \frac{1}{2} \rho dV \lambda^2 \omega^2 \sin^2 (t - \frac{\lambda}{V_s}) \quad \text{(3.1.2)}
\]

This the total energy should be

\[
dW = \rho dV \lambda^2 \omega^2 \sin^2 (t - \frac{\lambda}{V_s}) \quad \text{(3.1.3)}
\]

For the energy per unit volume i.e. energy density in each cycle can be written as

\[
\overline{W} = \frac{dW}{dV} = \rho \lambda^2 \omega^2 \sin^2 \left(t - \frac{\lambda}{V_s}\right) \quad \text{(3.1.4)}
\]

where the average value of \( \sin^2 \left(t - \frac{\lambda}{V_s}\right) \) is equal to 1/2, thus the mean energy density is

\[
\overline{W} = \frac{\rho \lambda^2 \omega^2}{2}
\]

Thus we can summarize the following:

1. The energy of a wave field differs somewhat from that of simple harmonic motion, because it is always the sum of both kinetic and potential energy of the same phase, i.e. it may reach to the maximum or minimum at the same time.

2. Wave energy is directly proportional to the amplitude square \( (A^2) \) and circular frequency square \( (\omega^2) \). Therefore, these parameters may have great influence on the wave energy.

3. From the wave field point of view, the iterated sum of amplitudes of two waves may cause extremely intensified interferences of waves as shown in Fig. 3.1. It is obvious that at time \( t_3 \) and \( t_4 \) the energy of wave motion is almost overwhelming. In case of seismic wave action, the energy exerted by interference will be much intensive due to stochastic wave action.

4. Conclusion

The character of wave movement on a site forming a wave field is so important that it may help us to understand how ground damages were on a site in a future event.

Different sequence of seismic effect comes from different pattern of wave field and in turn different wave field mainly depends upon different terrain condition. By the aid of such consequences, it can be expected that the terrain evaluation or engineering geological investigation can be fulfilled with furthermore consideration of siting a project or designing
soil foundation.

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References


COLLATERAL EFFECT AND BASIC PATTERNS OF SEISMIC LIQUEFACTION

LES EFFETS ASSOCIES DE LA LIQUEFACTION SISMIQUE ET LEURS TYPES ELEMENTAIRES

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ABSTRACT

Field liquefaction invoked in plain zone by strong earthquake shows a very complicate manner both in plane and in space. Besides sand boiling, which happens very commonly, large scale ground settlement, mass landslide and gravitational rupture are usually produced simultaneously. All these are considered as direct or following damage effects which form various permanent ground deformations. This article, used the case of Tangshan, China, as an actual example, classified seismic liquefaction into five patterns with their different condition of embodiment and characteristic of seismic damage discussed. A correct base is provided for our assessment of seismic engineering geology in macroseismic plain zone and planned the regional a seismic work. Seventeen typical photos of various liquefaction patterns are attached.

ABSTRACT

La liquéfaction de site résultant du macroseisme dans les zones de de plaine se montre d'une manière complexe en plan comme en espace. A coté du bouillage de sable tres connu, s'apparaitent simultanément l'affaissement de la surface de grande etendue, le glissement de terrain, la rupture gravitationale. Ces phenomenes consideres comme les effets de dommage directes ou additionnels du seisme forment les differentes deformations permanentes du terrain. Sur la base de l'analyse du seisme a Tangshan de Chine, ce papier explique les conditions personnifies et les caracteristiques de dommage des effets de liquefraction sismique qui sont divises en cinq types elementaires. C'est un point de depart correct pour l'évaluation de la geologie de l'ingenerie asismique dans les zones planes en macroseisme, et la planification des travaux para sismique regionaux. Il presente 17 photos de liquefaction de differents types.

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The earthquake of July 28, 1976 in Tangshan, China, proved that the field liquefaction invoked by shock expressed with very complex manners both in plane and in space. Besides the very common sand-boiling, large scale ground settlement, gravitational rupture and mass slump are also produced. All these are considered as a kind of direct or accompanying damage effect of seismic liquefaction which form various patterns of permanent ground deformation. Every different liquefaction pattern has its own different formation mechanism and destructive effect. It is a result of comprehensive factor and comprehensive influence implying stress field of earthquake, deposit structure of strata, properties of physical mechanics, hydrogeology and geomorphic condition.

According to the ground deformation and destruction and destruction manner caused seismic liquefaction could be classified into five basic patterns as shown in Table I. Through the corresponding geomorphic units of the various liquefaction distribution it is very obvious that the distribution is under the control of geomorphic units. (Table I, II)

1. Not Liquefied

Strata and sediment facies are internal causes of field liquefaction formation. As it is well known, the consolidation of strata usually enhanced along with the increment of geological age, namely the older the strata deposit age and the bigger the consolidation, then the smaller the possibility of occurring seismic liquefac-
	ivation. In Tangshan area, all the Quaternary, except the Holocene deposit, are under overconsolidated state. Its liquefaction capacity is basically exhausted. Therefore, field liquefaction occurs only in Holocene riverbed facies, shore riverbed facies, littoral plain facies and lagoon facies deposit and the corresponding geomorphic units. This law has been proved by the Tangshan event.

(1) Non-liquefied Zone

Composed of bare or shallowly embedded bedrock or ancient Quaternary residual deposit and slope deposit of non-liquefied strata, distributed over the low massif and hill area in the north part of the plain.

(2) Fossil Liquefied Zone

As field liquefaction is an important product of the plain area under earthquake, there is a distinct sequence in time for liquefaction formed in every earthquake in the past. Used as a kind of geological trace this liquefaction could be well preserved in strata profile formed in various geological periods. Quite many fossil liquefaction occurred in strata of median Pleistocene and upper Pleistocene in peneplain of the north part (early alluvial fan deposit or fluvial terrace of Luan He river). The peneplain of Tangshan city area, except the recent riverbed alluvium zone of Dou He river, is non-liquefied "blind zone" formed by fossil liquefied zone. No new field liquefaction are produced during the Tangshan event though it is within the strong earthquake area of magnitude XI
### Table I: Districts of Basic Patterns of Seismic Liquefaction in Plain Zone of Tangshan, China

<table>
<thead>
<tr>
<th>Pattern of district</th>
<th>Symbol</th>
<th>Pattern of sub-district</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Not liquefied</td>
<td>I</td>
<td>Non-liquefied</td>
<td>Monadnock of quaternary remanent deposit and slope deposit</td>
</tr>
<tr>
<td></td>
<td>I</td>
<td>Fossil liquefied</td>
<td>Old Quaternary stratum stratum, liquefied many times in history, in overconsolidated state due to densification led by vibration, will no more be liquefied</td>
</tr>
<tr>
<td>Liquefied settlement</td>
<td>II</td>
<td>Regional settlement</td>
<td>Found in littoral plain, young marine deposit loose structure, shallow ground water table. Large area of settlement due to dissipation of soil pore pressure after earthquake</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>Local settlement</td>
<td>Local uneven ground settlement caused by sandboiling after seismic liquefaction</td>
</tr>
<tr>
<td>Liquefied sandboiling</td>
<td>III</td>
<td>Slight</td>
<td>With occasional sandboiling, no regularity in surface distribution, no apparent ground deformation</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>Moderate</td>
<td>Mainly scattered, occurred in large area and in uniform manner</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>Severe</td>
<td>Whirling and strip pattern, in large area, with many ground cracks</td>
</tr>
<tr>
<td>Liquefied slump</td>
<td>IV</td>
<td>River bank</td>
<td>Large scale rupture and gravitational slump occurred along banks of recent rivers or ancient river tracks</td>
</tr>
<tr>
<td>Liquefied rupture</td>
<td>V</td>
<td></td>
<td>Large area of uneven settlement at the upper part of the slump zone, mostly occurred together with gravitational slump</td>
</tr>
</tbody>
</table>

### Table II

- Not liquefied
- Liquefied sandboiling zone
- Liquefied settlement zone
- Liquefied slump zone
- Liquefied rupture zone

- low massif and hill
- fossil liquefied
- liquefied-scattered pattern
- liquefied-netted pattern
- liquefied-whirling pattern
- liquefied-strips pattern
- inclined plain
- peneplain
- regional settlement
- local settlement
- bank slump
- ground slump
- settlement rupture
- slump rupture

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2. Sand-boiling

Sand-boiling is a direct and basic manner of seismic liquefaction. Sand-boilings of various macroscopic patterns were formed in the plain after the Tangshan event. They have different conditions of formation and different mechanical properties. According to the geometrical figure of tracks, they could be classified as follows:

(1) Scattered Star Pattern

Usually occurred in:
(a) Abandoned river bed zone in mid-aged alluvial fan of Luanhe river. The riverbeds here of the earlier period (changed course already) are rather level and straight. There is a tendency that river course move parallelly southeaswards. Ground feature is flat, simple, a long way off recent ground surface hydrologic network.

(b) Zone of homogeneous strata structure with shallow embedment of liquefiable sand layer or rightly exposed.

(c) Zone which has no significant variation of ground water table within large area.

Liquefaction of scattered star pattern show a feature of unevenly distributed white specks which are sand boiling mounds. Their sizes are reflected by the diameters ($D_L$), the distances between the mounds ($L$) represent their densities. The duration time of horizontal seismic wave and the divergence of seismic coefficients are shown indirectly by the sizes and densities of sand mounds. The longer the vibration duration and also the longer the duration of liquefied sand-boiling, then the diameters of sandboiling mounds are bigger as a result. The densities and distances of sand mounds roughly represent the embedding depth of liquefied layer and the amount of loading pressure. With the prolongation of vibration the sand water, which has high water head, at the vicinity of sand-boiling will concentrate rapidly towards low pressure zone and burst forth in succession along the sand boil path. Therefore, no additional sand-boil will be found within a quite large area. A zone of deep embedment of sand layer and large space between sand-boil is formed. When the sand layer is embedded not so deep, then liquefied layer will burst forth out of ground surface with no high pore water pressure required. Under such circumstance the space between sand-boiled is small, or link together into a whole piece. It could be subdivided into two patterns according to the space and diameter of sand-boil mounds.

(a) Scattered Pattern (photo No.3)

$D_L \approx$ about 30 - 50 m, $L > 100$m; the depth of embedment ($H$) of liquefied layer is generally more than 5m.

(b) Dense Pattern (photo No. 4)

$D_L \approx 10 - 20$m, $L \approx 50$m.

$H < 2$m.

(2) Network Pattern

Usually occurred in side river and larger riverbed area. As the geomorphic feature of riverbed and sedimental
structure are complex, seismic ground waves are influenced by the discontinuous boundaries of ground formed by the cross section of riverbed during transmission, which produce functions of reflection, refraction, focus and stationary wave; make the amplitude of ground movement at the structural surface of riverbed sediment changed obviously. Therefore, it could be subdivided into two patterns:

(a) Pulse Pattern (photo No. 5)

Occurred in certain "U" shaped river bend area. The direction of pulse bely is generally parallel to the direction of riverbend axis, the space depends upon the structural surface of sediment formed in riverbed when the course is changed as well as the wave length of ground wave.

(b) Tree Branch Pattern (photo no. 6)

This is a different kind of pulse pattern formed by the inhomogeneity of local stratum. It usually occurs beyond the focus of reflect wave of semi-circular riverbend.

(3) Whirling Pattern

Formed in the inner sides of free river bend group. The sediment structure and condition of medium boundary are often very complicated. When earthquake occurs, during the transmission of one set or several sets of horizontal ground waves, the reflected waves from the medium boundary in different direction may meet with different phase, or form whirling figures when there is an inclined top plate or bottom plate in the liquefied layer. By the geometrical shape of the figures they could be classified into two patterns:

(a) Broom Pattern (photo No. 7)

Occurred on the convex banks of rivers. The reflective waves cause angle during meeting because the shape of riverbend is very simple. The direction of convergence is in the direction of meeting of vectors.

(b) Eddy Pattern (photo No. 8)

Occurred in the inner side of riverbend group. The meeting of several vectors caused the ground turning in the same direction and formed closed concentric circles from the centre expanding outward.

The track of whirling pattern usually are multi-whirling layers around one axis. The amount, radian and condition of enclosure illustrate indirectly the intensity of ground movement and the resultant angle of wave vectors.

(4) Strip Pattern (photo No. 9, 10)

Generally occurred in:

(a) Level land in the front part of inclined plain.

(b) Young alluvial fan formed by sediment of late Pleistocene and Holocene.

(c) Convex bank of free riverbend with variable banks.

Its feature is shown by countless closely arranged sandboils and roughly parallel are strips. The radian and plane extension are not uniform, some in bundle shape, some nearly like brooms. The apparent distinction between tracks of strip pattern and whirling pattern are:

(a) Place extension of strip pattern is much bigger than that of whirling
pattern.
(b) Radius of curvature and width of strip pattern are much bigger than that of whirling pattern, too.
(c) Not limited by geomorphic boundaries of recent rivers, many strips run across recent riverbeds.

3. Seismic Settlement
Seismic settlement is a vertical ground deformation caused by liquefaction. Mostly is uneven. As the scale and amplitude of deformation invoked by seismic settlement are under the control of many complex factors, it might be classified into two patterns:

(1) Local Seismic Settlement (photo No. 11, 12)
A kind of accompanying effect invoked by sandboil. Soil structure of sand layer being densified and pore pressure are enhanced after earthquake. Pore water under high pressure will gush out of ground surface together with a large amount of sand and cause the ground settled down. The rate of settlement nearly equals to coefficient of volume compressibility, namely,

\[ S_0 = W_0 = n_1 - n_2 \]

where  
- \( S_0 \) -- rate of settlement (%)
- \( W_0 \) -- coefficient of volume compressibility (%)
  (ratio between pore water drained and its total volume)
- \( n_1 \) -- pore ratio of sand before earthquake (%)
- \( n_2 \) -- pore ratio of sand after earthquake (%)

The magnitude of rate of settlement depends upon the density of sand layer before earthquake and the intensity of earthquake, while the latter is determined by surface acceleration (\( \alpha \)) and the duration of vibration (\( t \)). The bigger the coefficient of volume compressibility, the longer the duration of vibration and time of liquefied infiltration, then seismic liquefaction becomes more intense and rate of settlement (\( S_0 \)) become higher.

Based on theories related to soil mechanics and the principle that hydraulic slope varies in accordance with depth and time, it is known that the time duration of pore pressure dissipation and variation of hydraulic slope are mainly determined by three factors: thickness of liquefied layer, volume compressibility and permeability coefficient. Since rate of settlement depends upon the difference of pore ratio of sand layer before and after earthquake, large amount of ground water are squeezed out from the sand layer and dissipated over the ground followed by densification of sand layer and settlement.

There are many case histories of local settlement in Tangshan event. The destruction of many buildings, large portion of railway and highway embankment and bridges are mostly caused by this reason and the foundation sank and tilted unevenly. For example, the railway from Chu Ge Zhuang to Tou Zi Tou (\( K_1 + 350 - 3 + 500 \)), occurred uneven subsidence in wave shape, with a maximum settlement of 2m, owing to general liquefaction of old course of Yi He river.

Local settlement also widely dis-
tributes over littoral plain. As the strip shaped settlement pit, 5m wide, 200 - 300 m long, over 3m deep, found in Xi He Liu Shu Qu Cun village after earthquake; the pit formed in Yao Juan Cun village, Wang Tan commune, Le Ting county, after earthquake was more than 10 m wide and 30 m long. A single story 5 - room house in the village sank into the ground as a whole with its roof of 60 cm. under ground level. The settlement was more than 4 m in depth. Testified by drilling test, it is the sudden change in the structure of shal- low ground layer that causes the above mentioned local settlement. The foundation failed entirely after seismic liquefaction and formed large scale settlement. (Fig. 1)

A ring-shaped line of level surveying (repeated) made after the earthquake from Tangshan city southward to Ninghe, Nanpu, Baige Zhuang, Leting, Luanxian showed the settlement of littoral plain is bigger than that of slant plain (Fig. 2). A settlement belt of about 1200 km², with its centre in Ninghe county, is formed after the event. The settlement is 1.46 m in the central part. (Fig. 3) The location of Fuzhuang commune, east of Ninghe county, is within coastal area with a seismic intensity of IX and Juzhuang bridge settled wholly after the quake.

The settlement was up to 2.6 - 2.9 m with an area of 1.5 km². There is a large ring-shaped fracture at the boundary of the settlement area. Some of the fracture were in the width of 1 - 2 m, but the damage to buildings within that area was very slight.

4. Slump

Slumping on river banks, slopes and boundary zone of underground mining, owing to loss of stability of soil by seismic liquefaction, is a very common phenomenon, which is a kind of permanent deformation caused by lose of shear strength of soil. There are two patterns of slump in the Tangshan event:

(1) River Bank Slump (photo No. 15)

This is the most common type. In plain area with many rivers there might be slump in various scales, which is determined by condition of soil deposits and feature of riverbeds, caused by seismic liquefaction.

Slump of river banks made the soil of one bank or both banks move towards
Fig. 2. Curve of repeated ring-shaped level surveying, Tangshan - Nanpu - Luanxian

Fig. 3. Large area settlement - vertical deformation isogram of Ninghe district after the quake

the centre of riverbed and uplifted the riverbed(191,530),(585,839) and narrowed the river course and blocked it up. For instance, at Zhao Tian Zhuang of Nu Zhi Zhai commune, south suburb of Tangshan city, the banks of Dou He river slide to a distance of 150 m and blocked the river course seriously. At the Fourth Hospital of

Tangshan the flood detention dam subsided, moved toward the river centre and the riverbed was narrowed to only half of its original width and entirely blocked after the earthquake.

Bank slump are very harmful to flood detention dam and bridges. Take Dou He river as an example, according to
official statistical data, the damage of highway bridges in this district is:
entirely destroyed 16 %
seriously destroyed 20 %
slightly damaged 20 %
in good condition 44 %

Most of the damaged bridges are related to slump and local settlement invoked by seismic liquefaction. The slide distance of river bank is decided by the vertical height from ground level to the centre of the river. Assume the height to be \( h \), the slide distance to be \( S \), then \( S = f(h) \)

According to the shape of river bend, the Dou He river could be classified into three categories: straight river \( S_1 = 10h \), general river bend \( S_2 = 10 - 20h \), river bend group \( S_3 = 20h \).

(2) Ground Slide (photo No. 16)

Not often occurred at present. For example, the old course of Hai He river nearby the Second Workers Sanitorium and Railway Sanitorium in Tianjin liquefied after earthquake. One side of the Second Sanitorium slid horizontally 1.5 m and caused a fall head of 0.8 m. Graben happened in the west yard. The low lime bank in Tianjin Alkali Factory was 2000 m long from east to west and 1000 m wide from south to north and covered an area of 151 hectare. The south part of the lime bank slid southeastward about 240 m, with fall head 5 - 15 m, after earthquake. Graben fracture was formed in the central part. Both slides moved apart horizontally 30 m.

5. Fracture

Fracture is a product of seismic liquefaction. Those mentioned here are gravitative fractures in concern with liquefaction. There are two patterns:

(1) Settlement Fracture (photo No. 17)

This pattern of fracture is mostly concerned with ground settlement of sand boiling. It is a kind of gravitative fracture invoked by the impact of water pressure to the overburdened layer. During the earthquake and uneven settlement after sand boiling. Therefore, the distribution of fracture could be traced by the extension of the track of ground linear liquefaction.

(2) Slump Fracture

This is gravitative fracture formed by liquefied slide, mostly in the form of arc or discontinuous line, and parallel to the axis of riverbed. Its size is connected with the shape of river bank, structure of soil stratum of the bank.

CONCLUSION

Classification of patterns for associated effect of seismic liquefaction will be helpful to the analysis and prediction of similar pattern of liquefaction to be occurred in various geological and geomorphic unit and its damage character in the future earthquake. It can also be used as a base for seismic precaution.
REFERENCES

Photo No. 1. Fossil liquefaction in Qa stratum, Feng Run district. The horizontal stratum is floated by water pressure and deformed.

Photo No. 2. Fossil liquefaction on cross section of stratum in Feng Run district. The horizontal stratum is floated by water pressure and formed into gear shape.

Photo No. 3. Liquefaction of scattered star pattern sand-boiling (SLANT PLAIN AREA)
Photo No. 4. Liquefaction of scattered star pattern
(LITTORAL PLAIN)

Photo No. 5. Liquefaction of vein pattern

Photo No. 6. Liquefaction of Tree Branch pattern
Photo No. 7. Liquefaction of Broom pattern

Photo No. 8. Liquefaction of Vertex pattern

Photo No. 9. Liquefaction of Strip pattern
Photo No. 10. Liquefaction of Arc pattern parallel to river bed in riverbed area.

Photo No. 11. Local seismic settlement liquefaction (Pavement of Luan Xin highway settle).

Photo No. 12. Local seismic settlement liquefaction in Wang Tan primary school, Le Ting county, a big tree settled down 5 m, with only its top above the ground.
Photo No. 13. Liquefaction of Regional Seismic Settlement: the average seismic settlement is 3 m in Fu Zhuang, Ninghe.

Photo No. 14. Large settlement slump on river banks of Luan He river.

Photo No. 15. Slump liquefaction: alkali slump in Tianjin Alkali Factory moved horizontally 240 m after earthquake.
Photo No. 16. Slump fracture:
The river bank slumped, embankment fractured, at the middle part of Liu Jia Zha.

Photo No. 17. Seismic settlement fracture:
The fracture at the west side of old Luan He river, Xin Zhai Zi is 1,600 m long.
APPLICATION OF THE CONCEPT OF "SAFETY ISLAND" RELATIVELY STABLE LAND (ROCK) MASS ... TO THE SITE SELECTION OF A NUCLEAR POWER STATION IN GUANGDONG PROVINCE

ABSTRACT

The definitions of regional stability and "Safety Island" are discussed in this paper. The purpose of studying regional stability is to select relatively stable areas as bases and sites of engineering construction. "Safety Island" is a common expression for a relatively stable land (rock) mass, denoting a land (rock) mass whose tectonic, magmatic, hydrothermal and seismic activity, regional physiogeological processes and engineering activities are now slight in an active structural belt and whose structure is intact. The paper proposes qualitative indications and quantitative indices for evaluation of regional stability as well as the assumed coefficient of regional relative stability.

Finally, taking the site selection of a nuclear power station in Guangdong for example, the active structural belt and the relatively stable land (rock) masses in the area are discussed. The relative stability of three rock masses and two land masses is evaluated on the basis of their characteristics. If comprehensive studies of regional stability could be carried out in China's vast coastal areas so as to find more relatively stable "Safety Islands", then more reliable scientific basis would be provided for selecting the site of nuclear power station in such areas.

ABSTRÁIT

Cette communication a exposé la stabilité régionale et la signification de "l'Ile de Sécurité". La recherche de la stabilité régionale a pour but de choisir des régions relativement stables comme des bases et des sites de construction d'ouvrages.

"L'Ile de Sécurité" est une expression couramment utilisée pour les masses de terrain (et les masses de roche) relativement stables situées dans la zone tectonique active et où les activités tectoniques, magmatiques, hydrothermales et sísmiques, les effets géo-physiques régionaux et les activités des travaux humains sont faibles, et leur structure est intacte. Dans cet article sont présentés les marques qualitatives et les indices quantitatifs de l'évaluation de stabilité régionale ainsi que la conception de l'évaluation du coefficient de la stabilité relative régionale.

Enfin, citant l'exemple de la sélection du site de la centrale nucléaire
Introduction

At present, different countries use different earthquake and fault safety criteria for siting nuclear power stations. The reason for this is closely related to the degree of stability of the crust and its surface where these countries are situated. Even in the same country, different regions show different modern tectonic activity due to different crustal structures and structural positions where these regions lie. The mechanical application of certain "criteria" or "regulations" in disregard of such differences will bring adverse effects to the siting of nuclear power stations. Therefore, it is of very important strategic significance to make a comprehensive analysis and evaluation of regional stability of a site-selection area and its vicinity from a seismic-geological and engineering-geological point of view, in connection with the actual geological conditions of China and with reference to the site-selection experience abroad.

Meanings of regional stability and "Safety Island"

So-called regional stability refers to the degree of stability of the present-day crust and its surface under endogenic and exogenic agents (dominantly the endogenic agent) in an engineering construction area and the mutual action and influence between such stability degree and engineering structures.

The earth has been constantly moving and changing since its formation; therefore, "activity" is its inexorable state and trend. But in a certain period and in a certain area, under certain stress conditions, the crust of a local area may be in a relatively stable state. For this reason, the study of regional stability must start with the study of activity; to put it in concrete terms, in areas for potential sites, it is necessary first to study tectonic activity, seismic activity, magmatic activity, hydrothermal activity, physico-geological processes and man's engineering activities, and then a comprehensive evaluation of regional stability can be made.

The study of regional stability is aimed at selecting stable areas as bases and sites of engineering structures. As early as the early 1960's, Prof. Li Susung (J.S.Iee), the famous Chinese geologist, had advanced the important argument of selecting relatively stable "safety islands" in the active tectonic belt as bases of engineering structures. This argument is now still guiding our study of regional stability.

The name "safety island" is not a technical term, but indeed very vivid and common. In fact, it is a synonym of relatively stable land (rock) mass. From a geomechanical point of view, the structural features on the surface of the crust may be macroscopically classified into fold (fracture) zones and land (rock) masses. Relatively, the former is active, while the latter is stable. During long periods of geological development, however, fold (fracture) zones and land (rock) masses are by no means unchange-

*Tang Changhan, Yi Mingchu and Dong Zhiguo took part in part of preparation work of this paper.
able. The early-stage fold zones may become rigid land (rock) masses contained within the late-stage fold zones. On the other hand, the early-stage land (rock) masses may also undergo intense late-stage tectonic movements and become active. The process of such mutual transformation involves a lengthy geological period, but a cycle of modern seismic activity (from the intensely active period to the quiescent gap) can be completed during the human history or during the engineering lifetime. Therefore, in studying seismic activity, it is necessary to study the law governing the changes and development of earthquakes in terms of time, space and intensity. As far as fold (fracture) zones are concerned, they may be further divided into secondary structures: uplifts, depressions and land (rock) masses within zones. Downfaulted basins and valleys that have formed since the Cenozoic tend to be controlled by active fractures and become zones of disturbance in the present-day crust, whereas uplift zones that rise over large areas are relatively stable. Of course, land (rock) masses within depressions and uplift zones are normally more stable than fold (fracture) zones themselves.

So-called relatively stable land (rock) masses refer to areas where modern tectonic, magmatic, hydrothermal and seismic activity and regional physico-geological processes are relatively slight and the crustal structure is relatively complete. According to their origin and structural positions, such land (rock) masses may be classified into several types. For example, according to the structural positions, they may be classified into land (rock) masses within, between and marginal to zones. According to their origin, they may be classified into ancient basement-type land (rock) masses, tectonic land (rock) masses (e.g., the nuclear column and horse-shoe-shaped basin of an epsilon-type structure), magmatic land (rock) masses and metamorphic land (rock) masses.

The study of the formation and development of different land (rock) masses, their medium and structure characteristics and their present-day activity is of major theoretical and practical significance for regional stability evaluation and site-selection in areas of engineering construction.

Criteria for judging a relatively stable land (rock) mass and suggestions for quantitative stability evaluation

The stability of a land (rock) mass depends upon its size, form and structural position, medium and structure characteristics, tectonic history (especially the Neoid tectonic history), modern geomorphological evolution and present-day tectonic stress field. The criteria for determining a relatively stable land (rock) mass may be summarized as follows:

1. With regard to tectonic units, it belongs to a land mass, rock mass or rigid fold zone.
2. With regard to media and structures, it is composed of elastic or quasi-elastic, relatively homogeneous and continuous media, exhibiting a massive or very blocky, thickbedded structure. The weak interbeds and structural planes of weakness in the rock mass are not developed.
3. With regard to tectonic activity, there is no active fault or capable fault that penetrates it and it belongs to the present-day tectonic stress field with a low stress value.
4. With regard to seismic activity, there is no strong earthquake-generating fault that penetrates it; the attenuation distance from a regional strong earthquake-generating fault and a zone of fracture compounding is great enough not to cause earthquakes of high intensity. It belongs to a region of low intensity, where microseismic activity is rare and seismic effects are not conspicuous.
5. There is no modern magmatic or volcanic activity, and hydrothermal activity is not intense either.
6. Regional physico-geological processes such as landslides, ava-
lanches and mudflow are not strong or only restricted to some segments. 7. There is no man-made destructive engineering-geological events, for example, land subsidence and col- lapse caused by excessive pumping of fluid deposits and earthquakes induced by deep-level water injection and reservoir filling.

The above-mentioned criteria are only qualitative. To make a quanti- tative evaluation of regional sta- bility, various measuring indices and related parameters affecting regional stability are also required, e.g.:

1. Index of fault movement. It is represented by the rate of fault movement (f/m). If the data of long- term observation of displacements on faults are available, the yearly rate of movement may be directly obtained. In the case of absence of the above-mentioned data, D.B.Slemmons used the recurrence intervals of different magnitudes of seis- mogenetic faults to calculate the rate of movement for faults. The magnitude-recurrence relation is that: within the same recurrence interval, the greater the magni- tude, the higher the rate of fault movement; while for the same magni- tude, the longer the recurrence interval, the lower the rate of movement. He classified the rates of fault movement into high rate (AA=10.0 cm/yr, A=1.0-10.0 cm/yr, A=0.1-1.0 cm/yr), moderate rate (B=0.01-0.1 cm/yr) and low rate (C=0.001-0.01 cm/yr).

2. Index of seismic activity. In the siting criteria for nuclear power stations, special attention is paid to the determination of the para- meters of seismic activity, and it is necessary to determine the Operat- ing Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE). According to the empirical data, OBE ≮ 2 SSE.

It can be found from above that the key parameter for determining the OBE and assessing regional stability is the epicentral intensity (I0) of the maximum probable earthquake (MPE) in the same seismotectonic province.

The maximum felt intensity (I) at the site-selection area after the attenuation of the epicentral in- tensity may be obtained from the following formula:

\[ I = I_0 - \lg \left( \frac{A^2}{H^2} + 1 \right) \]

where A is the distance from the epicenter to the site-selection area or the nearest distance from the possible earthquake-generating locality after the MPE shifts, H is the focal depth and S is the coef- ficient of intensity attenuation. The maximum felt intensity thus obtained should be adjusted appro- priately in connection with site media, structural conditions, ground- water character and geomorphology so as to fix the maximum felt inten- sity of the site.

3. Index of stability affected by regional physico-geological proce- sses. In geological-geographical regions in different climatic and tec- tonic zones are developed regional physico-geological processes of different types and intensities. But in mountain regions as well as lake- and sea-shore regions, move- ments of masses of rock or earth are commonly developed, which are produced under the combined effects of endogenic and exogenic agents. The instability of mountain masses expressed by large-scale landslides and avalanches often bring devastat- ing disasters to engineering structures. In certain intensely active structural zones, the above- mentioned physico-geological pro- cesses tend to occur in zones, and affect the stability of a region. The index in mountain mass stabil- ity evaluation is the coefficient of mountain mass stability (Km). It depends upon the media and struc- ture of a mountain mass, especially the shear strength of weak interbeds and weak structure planes, the relation between the sliding mass and the antisliding mass and the dynamic factor causing insta- bility of the mountain mass. In areas with intense tectonic and seis- matic activity, instability caused by seismic effects is a problem.

that should receive still more attention.

4. Index of quality of mountain and rock masses. It affects the stability and anti-seismic performance of mountain and rock masses. The main index is their quality coefficient \( Q \), which depends on the intactness of mountain and rock masses and may be expressed by the fissure density \( K \). Generally, the intactness of a rock mass with a high fissure density is poor; conversely, the intactness is good. Besides, the quality coefficient \( Q \) also depends on the compressive strength \( f \) (or expressed as the coefficient of firmness \( f' \)), i.e., \( f=R/100 \) and the weathering coefficient \( w' \), that is, the quality coefficient of mountain and rock masses

\[
Q_{s} = \frac{1}{K} f' \cdot w
\]

From the four parameters mentioned above, a function formula of the relatively regional stability coefficient \( q_{s} \) may be developed, i.e.,

\[
q_{s} = (f' \cdot I_{q} \cdot K_{h}) \cdot K_{h}
\]

The areas with different degrees of activity are characterized by different relatively regional stability coefficients. According to this coefficient, in connection with geological and seismic conditions, an evaluated region may be subdivided into several areas with different degrees of stability. For example, in areas where regional physico-geological processes are not well developed and fault movement is not prominent, the relative regional stability coefficient mainly depends upon the intensity of the MPE and the quality coefficients of mountain and rock masses within the plant area.

As the factors affecting the regional stability are numerous and complex, the above-mentioned views are not mature and await further intensive consideration so that the regional stability evaluation might become quantitative step by step.

Preliminary determination of a relatively stable land (rock) mass in the site-selection area for the Guangdong nuclear power station

The main site-selection area for the Guangdong nuclear power station is located in the coastal zones of southern Guangdong. The structural position belongs to the southwestern segment of the Lianhuashan active fracture zone of the second uplift belt of the Neocathaysian structural system. This zone, striking NE-SW, is composed of intense fracture bundles and an intervening dynamometamorphic zone. Two intense fracture bundles may be clearly distinguished: the north bundle belongs to the Wuhua-Shenzhen fault zone, and the south bundle is the Haifeng-Weilong-pinghai fault zone (Fig. 1). The former is larger in Fig. 1. Map showing the

![Map showing the site-selection area](image-url)

**Fig. 1**

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distribution of active fracture zones and land (rock) masses in the southwestern segment of the Lianhuashan fracture zone. 1- Cenozoic subsidence basin and trough; 2- Mesozoic fold-fracture zone; 3- Mesozoic granite mass; 4- Land mass of Paleozoic low-grade metamorphic clastic rocks; 5- Latitudinal fracture zone; 6- Early-stage Neocathaysian fault zone; 7- Late-stage Neocathaysian fault zone; 8- Northwest- trending fault zone; 9- Active fault zone; 10- Fault zone inferred from geophysical data; 11- Epicenter of an earthquake of magnitude ≤ 3; 12- Epicenter of an earthquake of magnitude 4-4.5; 13- Hot Spring, scale. It extends for about 300 km, and has been proved by the aeromagnetic continuation to be over 20 km in depth, most probably attaining the Moho (the measurement of transformed waves of natural earthquakes show that the Moho in the area is up to 20-23 km deep). The latter is over 100 km in strike length and shallow in depth, developed within a depth of 10 km as shown by the aeromagnetic continuation. In both boundaries, recent seismic activity is relatively intense. According to the historical seismic record, seven earthquakes of intensity VI-VII have occurred throughout the Lianhuashan fracture zone, of which only 2 were reported within 30-50 km of the site-selection area. The two fracture boundaries have a tendency to gradually converge in a northeast direction; and diverge in a southwest direction; therefore the stresses are relatively concentrated. Generally speaking, the tectonic activity, seismic activity and dynamometamorphism are strong in the northeastern segment, and they become gradually weak towards the southwest. In the north-south direction, the activity close to the Cenozoic continental and marine basins on both sides is strong, while that in the uplift area in the central Lianhuashan fracture zone is relatively weak. Apart from the Cambrian, Middle Devonian and Lower Carboniferous, the strata contained in the fold-fault zone also include Yanshanian granite, Late Jurassic volcanic rocks and Late Triassic-Early Jurassic clastic rocks. In general, the site-selection area in the uplift block in the central Lianhuashan fracture zone is relatively stable. This area is more than 280-300 km both east of the Nan'ao strong earthquake area and west of the Yangjiang strong earthquake area, 125 km north of the Heyuan strong earthquake area, and over 100 km south of the littoral deep fracture.

But as far as the compounding relationship of several structural systems and the indications of the Neoid tectonic activity and microseismicity in the area are concerned, it is necessary to look for a much more stable land (rock) mass as the site for the nuclear power plant within the above-mentioned relatively stable area. This mass is just the reduced scope of the "safety island". So-called reduction is for the purpose of keeping clear of those fracture with relatively slight indications of activity and those secondary fractures whose activity indications are not easy to ascertain for the time being. According to the abovementioned criteria for identifying a "safety island" and by making full use of the existing data on surface surveying, historical earthquakes, deep-level geophysical investigation, fault activity survey and recent micro seismic detection, the author et al. divided the site-selection area in the Lianhuashan active fracture zone into four Paleozoic land masses and four Mesozoic rock masses. Through comparison and evaluation, (a comparative table of relative stability characteristics of land (rock) masses has been omitted). Finally the relatively stable "safety island" has been determined. Its characteristics are as follows:

1. The plant foundation consists of fine-to coarse-grained biotite granite, belonging to a high-strength, homogeneous and sound rock mass.

2. The plant site is more than 20 km both north of the Wuhua-Shenzhen active fault zone and southeast of the Haifeng-Pinghuai active fault zone and over 5 km east of the minor capable hidden fault of the Central Islands.
Fig. 2. Photo illustrating the coastal slope of siting area which made up of intact granite mass.

3. Historical earthquakes are rare in the area; no earthquakes of Ms > 3 have occurred within 20 km of the plant site. The observations of the microseismic network from April to September in 1981 show that there have been no microseismic indications within 8 km of the site and that only five earthquakes of Ms < 1.4 have been recorded within 40 km of the site.

4. Data of the aeromagnetic continuation and transformed waves of natural earthquakes indicate that no major deep fracture has yet been found in the site area.

5. There are no such adverse physico-geological phenomena as modern volcanos, hot springs, regional landslides and avalanches in the site area and its vicinity.

6. The granite mass is intact, which is beneficial to radio-active residue and effluent disposal and storage.

In summary, we preliminarily maintain that the Yanshanian granite mass of sitting area is a structurally intact and relatively stable rock mass. Its discovery has provided safe and reliable conditions for the siting of the first nuclear power station in Guangdong. As at present various measuring indices have not yet been treated and presented, the quantitative evaluation of regional stability remains to be further studied.

Concluding Remarks

It appears that the application of the concept of the relatively stable "safety island" advanced by Prof. Li Suukuang and the qualitative and quantitative evaluation of regional stability of site-selec- tion areas for nuclear power stations on the basis of regional geological, seismic-geological and engineering geological conditions are an important subject in the phase of planning and site selec- tion. If a comprehensive study of regional stability can be carried out in wide coastal areas of China to find relatively stable "safety islands", then scientific grounds will be provided for the planning and siting of more coastal nuclear power stations and other major engineering works in China in the future. This can not only save investigation investments but also shorten the turnaround of construction and increase safety and reliability.

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AN OUTLINE OF THE EVALUATION OF GEOLOGICAL STABILITIES FOR NUCLEAR POWER PLANT SITE IN CHINA

UN PROGRAMME SOMMAIRE DE L'EVALUATION DE LA STABILITE GEOLOGIQUE DU SITE DE LA CENTRALE NUCLEAIRE DE CHINA

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ABSTRACT

Based on the recent siting experiences of several nuclear power plants situated in East China, Southern Liaoning and Guangdong province, the authors put forward an outline proposal concerning the evaluation of local and regional stability of nuclear power plant site under the particular conditions of China.

In China frequent earthquake has been observed and an abundant wealth of historical data has been accumulated for more than 2,000 years. Hence it appears that the evaluation of local and regional stability of nuclear power plant site should be centered on the identification of Basic Earthquake Intensity. Based on these grounds, the authors propose the basic geological siting principles of nuclear power plant and the methods on the assessment of surface faulting in site area, SSE and OBE.

ABSTRAIT

D'après l'expérience recemment obtenue dans le travail du choix des sites des plusieurs centrales nucléaires situés à l'Est de la China, au Sud de Liaoning et à Guangdong province, l'auteur a propose un programme sommaire concernant la stabilité locale et régionale du site de la centrale nucléaire sous les conditions concrètes de la China.

En China, sous les conditions où, les séismes se produisent fréquemment et pendant une longue de plus de 2,000 ans de riches documentations historiques du séisme ont été accumulées. On doit proceder en concent rant sur l'identification de l'Intensité de Base pour l'évaluation de la stabilité locale et régionale du site de la centrale nucléaire.

D'après cette idée, on a propose les principes fondamentaux sur le choix du site de la centrale nucléaire et les méthodes concrètes de l'action de failli à la surface près du site, du SSE et OBE.

Since 1974, the several nuclear power plant sites in the East China, southern part of Liaoning and Guangdong provinces have been selected and some useful experiences have been accumulated. The criteria of application for evaluation of local and regional stability of siting nuclear power plant

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is now under discussion. A principal outline is put forward herein.

1. BASIC CONSIDERATION

China is a much-seism country as it is located between the ring Pacific seismic zone and the Mediterranean-Himalayas seismic zone. Macro-seism occur frequently in this region. Hence, the major topic in evaluating the potential geologic hazards of nuclear power plant site is related to the seismotectonic activity and the regional stability of possible site region.

China is also a country with an ancient civilization. The earliest historical earthquake records might have been dated from 1831 B. C. with a history of more than 3,000 years. In most provinces and regions in China, more than 1,000 years of historical earthquake records including felt weak-earthquakes are available.

Since the 1950s in China, the seismologic work and the seismogeologic research have rapidly developed. Abundant data and results have been accumulated on the earthquake monitoring, seismogeology, geophysics, geodetic survey, earthquake engineering and a seismic design. Besides the Basic Earthquake Intensity (BEI) map of all China, some provinces or regions have published their own local map of BEI. All of these have provided convenience for siting nuclear power plant and evaluating the regional stability.

Hence, the siting of nuclear power plant in China can be made by collecting those regional geological and seismological data and may be studied according to the following principles:

(1) In the region of low BEI, far from macroseism active zone or seismic hazardous area.

(2) In the area of simple geologic structure and of the bedrock out cropping or shallow overburden.

(3) On gentle level land, a large amount of excavating or filling are not needed.

(4) The water supply is sufficient with convenient condition of the water supply and drain off.

(5) In area of a low density of population and there are not the large factory, mine, farm, grazing land and aquatics product etc. nearby, but the facilities for transport are needed.

2. SURFACE FAULTING

2-1 General Requirement

The presence of surface faulting or potential surface faulting at the site or within a specified distance from the nuclear power plant site is not permitted.

The surface faulting may be divided into two types, i.e. the slow creeping and rapid seismic faulting.

2-2 Creeping Fault

The creeping fault is present in a strong active tectonic zone and is quite rare in China. Therefore, it may be not considered. But for area of candidate site within or near macro-seism region attentive research should be taken on the fault movement history in Quaternary Period, if where an active evidence of fault has been observed since Neogene Period. The arrangement of precise instrumental observation should be made in the area to make sure whether it is a creeping one.

2-3 Seismic Fault

The seismic faults are quite common in China. Some of them may have a width of more than a hundred meters and extend more than a hundred kilometers in length. According to statistics of actual earthquakes, seismic fault may occur only when the magnitude of earthquake $M > 6$.

2-4 Capable Fault

A fault, which has potential to occur surface faulting is considered as the capable. In the case of China, the following distinguish marks of the capable fault can be taken:

(1) Fault with fracturing deposits since Miopleistocene ($Q_2$).

(2) Those faults, in which the earthquake of $M > 6$ had occurred, or the faults with the potential capacity of earthquake $M > 6$.

(3) The fault is of obviously tectonic relation with the capable fault, such as the branches of the capable fault, fault intersected or cross intersected or closely parallel with the existing capable fault which may be associated with each other in the depth.

2-5 Investigation of Capable Fault
In the planning phase of nuclear power plant engineering project in China, geological survey (scale 1:50,000) has been carried out for every candidate area with an extent of 8-10 km, in radius from the center of the site. Hence, the faults longer than 1 km, had been discovered in the geological investigation. In order to judge the fault is or not capable, the following data are required.

(1) The occurrence and scale of the fault and its tectonic association with the regional active fault should be investigated. If possible the depth of the faults and their tectonic relation in the depth should be researched.

(2) The formation of the fault and its history of movement, whether any macroscopic evidence of Neotectonic motion is presented.

(3) Exact data of the fault action and its relative displacement, especially those related to the earthquakes since the Miopleistocene should be investigated if it has macroscopic evidence of Neotectonic movement.

(4) The relationship between the fault and earthquake.

According to above data and analysis of the intense earthquake potentiality, we can determine whether the fault is capable or not.

3. THE SEISMIC GROUND MOTION ON THE SITE AREA

3-1 Basic Earthquake Intensity

The Basic Earthquake Intensity is an index used to divide and to assess a certain area which will suffer the maximum seismic ground motion. In the past, the BEI has no definite time limit. Recently, it has been defined that the BEI of a certain area is considered as the maximum earthquake intensity, occurring within the next 50-100 years.

It should be pointed out, although the BEI reflects the level of knowledge of men on the natural phenomenon of earthquake, the reliability of the BEI has increased with the advancement of seismology, but it is imperfect. Therefore, it is necessary to study further on the BEI of nuclear power plant sites.

3-2 The Determination of OBE

The OBE is the potential maximum seismic ground motion on the site area during the operating life of the plant for about 40-50 years. Obviously, it conforms with the concept of BEI. Therefore, in those regions in which the seismic data are abundant or the geological structure is simple and the earthquake intensity is lower, the OBE of the nuclear power plant may be directly apply the BEI on the BEI map. But if the site is adjacent to the macroseism active region with complex geological structures, than it is necessary to identify the reliability of BEI.

3-3 The Determination of SSE

The SSE is the ultimate safety parameter of an anti-seismic design for the nuclear power plant. It requires to apply evaluate the potential maximum earthquake intensity on site, which is caused by the maximum earthquake in the site area and in its adjacent region for a comparatively long period. The SSE used to determine by the seismogeological methods.

The major procedures are as follows.

(1) Determination of the tectonics and/br faults, in which the potential macroseism may occur.

(2) Estimation of the potential maximum earthquake magnitude (or intensity) of those seismic tectonics and/or faults.

(3) To assume the maximum earthquake which will occur along the nearest tectonic and/or fault to the site.

(4) To propose the maximum earthquake intensity in the site by selecting an appropriate intensity attenuation function.

3-4 Principles and Methods for Determination of SSE

The determination of SSE is similar the division into the earthquake intensity, in which the time factor is not to be considered. So the general methods and procedures may be applied as follows.

(1) Within the same earthquake hazardous region the maximum earthquake occurred in history may reoccur in the future.

(2) In the regions with similar seismic tectonic and same weak earthquake activity the potential capability
of macroseism may be same.

(3) The earthquake with VIII or more VIII intensity usually occur at large and deep faults and at its ends, corners or intersection points. The earthquake with VI-VII intensity mostly occur at large faults or within the Faulted Block. The correlation between single weak earthquake (Intensity < VI) and the tectonic is not obvious. But the regions with crowded weak earthquake conform to the active region of Neotectonic movement.

(4) Methods to determine the maximum magnitude or intensity of earthquake in the future:

a) The maximum magnitude of historical earthquake will be taken as the maximum earthquake magnitude in the future, if seismic data and geological data are abundant and conformation.

b) The maximum magnitude of historical earthquake may be taken as the maximum earthquake magnitude when the macroseism had occurred in history but the geologic evidence are not obvious.

c) An identical macroseism may occur on the other sections of the same tectonic zone if the macroseism occurred on it. This conclusion can be given on the basis of geologic data by the method of extrapolation.

d) In a seismic active period, in the region of crowded earthquakes with VI-VIII intensity the stronger earthquake in the future may occur.

e) When the geological conditions of macroseism zone, which is lack of historical earthquake data but frequent weak earthquake in the recent the maximum earthquake intensity in the future corresponding to the historical maximum earthquake intensity, which had occurred in the area of similar geological conditions, should be considered.

When the SSE of a concrete nuclear power plant is estimated, the application of the above-mentioned principles and methods should conjoin closely with other methods and empirical formula should be adopted in prediction, if enough and accurate data have available, in order to check each other.

3-5 Investigation of Prehistoric age Earthquake

"To extrapolate the future from the past" is an important principle of earthquake forecast. In China, under the much earthquake and abundant historic earthquake data for more than 2,000 years, the forecast of the seismic active tendency and the magnitude should be reasonable and believable according to the above-mentioned principle in the next 50-100 years (such as the OBE). But it is unreliable to extrapolate the earthquake in the future 1,000 or 10,000 years (such as the SSE). Therefore, investigation of prehistoric age earthquake and study on the prehistoric age seismic history is important in a fault or in a region. It may be achieved by the archaeologi- cal method and fault trench method.

3-6 The Maximum Ground Surface Acceleration and Time History

The macroseismic records by the strong-shockgraph and the data of near field macroseismic records are very few in China. The macroseismic records in all planning nuclear power plant area have not been obtained. Hence, the acceleration response spectrum and time history of SSE and OBE in the nuclear power plant sites may be determined by the IAEA Nuclear Safety Series No. 50-SG-S1 and No. 50-SG-S2 and the Regulatory Guide of USNRC for the time being.

4. THE SECONDARY EARTHQUAKE CALAMITIES

The potential secondary earthquake calamities on and near the site of plant should not occur during the possible maximum earthquake. These include:

1. The liquefaction, subsidence, tile, uplift, landslide and crack of the ground of site.

2. The collapse, landslide, lose-face of shore, deposit, flood (include flood caused by the dam burst during earthquake), surge and slope slide of fill etc.

Conservative risk analysis and dynamic stability analysis of these secondary earthquake calamities should be carried out, based on the acceleration response spectrum given from the surface motion study.

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ON THE DETERMINATION OF RAILWAY BRIDGE SITE IN ACTIVE FRACTURE ZONE

LE CHOIX DU SITE D'UN PONT DE CHEMIN DE FER POUR TRAVERSER UNE ZONE DE FAILLES ACTIVES

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Abstract:

By sumning up the recent seismic studies of China and applying the research results of seismotectonic, taking as an example the determination of bridge sites in active fracture zone along the Yanzhou-Shijiusuo Railway in Shandong Province and the Beijing-Qinhuangdao Railway in Hebei Province, the present paper deals with the analysis of tectonic stability and seismic intensity conditions of the field of railway bridge site, suggesting a classification of three lock types of active fault and providing a method of determining "safety island".

Résumé:

Selon les données de séisme moderne de Chine et les succès de recherche géologique de séisme, sur les exemples de deux sections de chemin de fer (celle de Yanzhou à Shijiusuo, Province du Shandong et celle de Beijing à Chinhuanjiao, Province du Hebei) qui traversent des zones de failles actives, on explique comment y trouver un site relativement stable s'appuyant sur les données de structure géologique et les enregistrements historiques de tremblement de Terre pour choisir la traversée d'un pont. On indique 3 types de fermeture de failles actives et la façon de trouver "Ile de sécurité".

Along active fracture zones, there are often rivers. When a railway has to be built across an active fracture zone in which long bridges are to be set up, the displacement of the active faults and the factors of earthquake occurrence should be considered in order to determine suitable bridge sites.

1. Influence of Active Fracture Zone on the Stability of Railway Bridge Site

This influence is reflected mainly in the following respects:

1. Displacement of exposed or near-surface active faults.

Two modes of fault movements can be subdivided: the creep and the sudden displacement. For example, in the area near Beijing, fault creep with annual displacement rate of 0.1-0.7 mm is observed. Some of creeps exhibit reciprocating nature, while others are obvious and will cause a greater value after occurrence for a long period. For example, in Sichuan Province, South-west China, along the Minjiang Fault, the maximum vertical displacement has reached 0.7 m in 17 years with an annual rate of 9-26 mm. During a strong earthquake, obvious and rapid
displacement would occur at the seismic fault. For example, the 1976 Tangshan earthquake of magnitude 7.8 (on the Richter scale) produced a maximum vertical displacement of 1 m and a maximum horizontal displacement of 2.5 m; the 1970 Tonghai earthquake of magnitude 7.7 in Yunnan Province produced a maximum vertical displacement of 0.5 m and a maximum horizontal displacement of 2.7 m. Obviously, if a bridge is located on an active fault with evident creeps and rapid dislocation, its long-term stability can not be secured.

2. Possible strong earthquake in active fracture zone

In China, 80% of the earthquakes of magnitude ≥ 7 and all of the earthquakes of magnitude ≥ 8 occurred in active fracture zones. The strong tremors of the earth's surface caused by earthquakes will lead to damage of buildings, which is more common and more extensive than that by dislocation of faults. For example, the 1976 Tangshan earthquake of magnitude 7.8, damaged on different degree 39% of the 193 railway bridges.

II. Active Faults and Their Lock Parts

The analyses of contact relations of Quaternary strata, landform features such as gullies, valleys, terraces and planation surfaces, and drainage changes are the basic methods for determining the activities of the faults in the Quaternary period, or even in recent time (Fig. 1). The geodetic measurements of present topography and faults and the analysis of historical and recent seismities data are important for obtaining evidence of recent activities of faults.

![Fig. 1. Expressions of activities of Quaternary faults](image)

Analysis of recent strong earthquakes in China with accurate geodetic data indicates that strong earthquakes mostly occur in the so called lock parts of active faults, i.e., at the ends, in the incompletely connected parts, or in the parts hindering the fault activities due to various factors. High energy can be accumulated in certain parts of active fault because of the locking effects. Strong earthquakes will occur when rapid fracturing takes place. Lock part is the most dangerous part of an active fault zone, where maximum displacement and epicenter of strong earthquake most probably occur. Therefore, analysis and identification of possible lock parts are significant.
for the determination of railway bridge sites. Three types of lock structures of active faults are preliminarily classified as follows:

1. The type of single large fault

It refers to large regional fault which has a large size and a long history of development, and tends to be the boundary of a geotectonic unit. Generally, its shattered zones are well connected and creep readily takes place at this fault due to repeated tectonic movement. Particularly in the part where cracking has already taken place, new strong earthquake is less probable. However, a lock structure may still be produced under the following conditions: The connection is incomplete locally; welding by intruded magmas in some parts; at the ends of large faults or main derivative faults; in parts of the large fault plane tending towards distortion; in the parts of main fault cross-cut by lateral small faults or interrupted by uplifts; at the turning section of an arc fault with maximum curvature; shattered zone of fault locked by the huge rock mass; etc. All these conditions may produce resistance to the free movements of faults, resulting in the formation of lock structures (Fig. 2 I).

2. The type of intersecting faults

It can still be subdivided into three subtypes: a. two faults of different trending intersected each other and on the intersection part a fault demonstrates convexo-concave structures on its fault plane or the intersection part was welded by injected materials; b. a sliding fault is constrained by another fault; c. two sets of faults meet but do not intersect each other. All these subtypes can produce lock structures, particularly during the changes in the directions of tectonic stress, they would intensify the activity of small faults and obstruct the movement of main fault, resulting in the formation of a lock structure (Fig. 2 II).

Fig. 2. Types of lock structures in active fault zones

3. The type of anastomosing faults with down-faulted basin

Along active fracture zone, especially at the intersecting locations, graben-type down-faulted basins are often formed, which are composed of relatively complex alternate fault blocks. At the boundaries of these basins abundant lock points which are controlled each other may emerge and high energy may be accumulated. Sudden
movement of any of these points may cause chain movement of other points and occurrence of a series of strong earthquake (Fig. 2111).  

In general, grand segments with greatest possibility formation of lock structures are main active faults, structure intersections and young down-faulted basins. Especially the poor connection of deep active structures which have deep structure background with the shallow structures is an important condition for the formation of lock structures. Therefore erecting bridges on these structures is not suitable because of their instability.

III. Determination of Bridge Sites in Active Fracture Zones

The locations of obvious displacement of fault are usually the epicenters. However, their harmfulness to engineering construction is different in nature. The displacement of fault can not be prevented in construction, but it has a rather small range of influence with a single factor; whereas earthquake may lead to a large range of influence through the travelling of surface elastic waves and has a lot of factors of influence. Nevertheless, such influence can be overcome by anti-earthquake measures to a certain extent. Therefore, the determination of bridge sites should be made according to the following principles:

1. A bridge should be erected on relatively stable block to avoid direct crossing active fault and kept away from lock part.

An active fracture zone usually consists of many faults. The main fault is the center of a fracture zone and the lock point is the dangerous part within it. Other big or small faults are different in activity, and the complete block among them is a relatively stable part which is most suitable for erecting a bridge. The investigations of a series of earthquakes occurred in China show that the damage caused directly by displacement of faults is limited in range. For example, the dislocation zones of exposed structural cracks resulting from strong earthquakes in Tangshan and Haicheng were 30-60 m in width, and the dislocation zones produced by other earthquakes were generally less than 100-300 m in width. Many buildings were located in pleistoseismic zone near the seismic fault, but most of them which were not built directly on the fault-displacement zone and had a solid foundation were not severely damaged. Therefore, it is of significance for the determination of bridge site to carry out detailed geological survey in the active fracture zone, distinguish and analyse the details of individual components of it.

The new-built Yanzhou-Shijius Railway in Shandong Province passes eastward across the well-known Chinese great Tancheng-Lujiang Fracture zone. This is an important active fracture zone in East China, where an earthquake of magnitude 8.5 took place in 1668. It trends NNE and consists of four paralled main faults (number F1-4). Exposures of newly-activated faults and obvious morphotectonic features are found frequently along it, which control the development of the Shuhe River system. Each main fault has an unmemented shattered zone of 20-100 m in width. Levelling and triangulation survey across the faults can indicate its obvious trend of anomaly and confirm its present-day activities. In this area there are many NW- trending faults (number F1-3) intersecting the NNE-trending faults, which are also Cenozoic activated faults. These two sets of faults converge and intersect each other, to form a new down-faulted basin (Fig. 3) in the late Pleistocene (q3), which reached the maximum subsidence in Holocene (q4), and continue to subside in recent times. Furthermore there are frequent micro-earthquakes along the main fault of the Tancheng-Lujiang and near the boundaries of the down-faulted basin.

Special explorations in above mentioned region show that a lock structure can be formed at any of the locations such as the main active fault and related larger fault, main intersection of major faults and young down-faulted basin. Therefore, bridge sites No. I, III, IV and V across or near the main fault and the young down-faulted basin are all rejected (Fig. 4). Bridge site No. II is on the block bounded on three sides by faults in which there is no active fault passing through. The distance between this site and the main active fault and the possible lock part is over one km, which considerably exceeds the width of the widest shattered zone of any fault in the study region (ranging from 60 to 110 m, with the maximum width of less than 300 m if the width of the influence zones on the two
Fig. 3. Geotectonic map of the segment selected for the site of the long Shuhe River bridge.

1. Contour of surface (m); 2. Isopach of the Quaternary strata (m); 3. Exposed area of bedrock; 4. Area in which the depth of buried bedrock is greater than that of the eroded datum in the lower reaches; 5. Active fault; 6. Common fault; 7. Late Quaternary young down-faulted basin; 8. Isolated hills on ascending wall of fault; 9. Location of micro-earthquake epicenter; and 10. Bridge site and its number.

Besides, the bedrock is shallow (0-5m) and the foundation of the bridge is comparatively complete and solid. Therefore this site is considered to be relatively stable, and suitable for the bridge.

Structural stability should be analyzed in combination of the variation of the tectonic stress field. For instance, site A of a certain bridge, as shown in Fig. 5, is located at the intersection of a NW-SE main fault (F1) of the Tancheng-Longjiang Fracture zone with a NW-SE fault (F2) and passes across an active fault. Due to the variation of the direction of principle compression stress in tectonic stress field, the intersecting pattern may be changed and a lock structure may be formed through the activity of the faults. Thus the bridge site is changed to site B.

Fig. 4. Geological section of the site of the long Shuhe River bridge.

2. A "safety island" should be selected for bridge through comprehensive and synthetic consideration of seismic intensity conditions of the field of
In addition to the displacement of faults, the strongly vibrating earthquake waves are also a destructive factor, which may cause even more extensive damage to bridges. The seismic intensity conditions of an site field which reflect the possible degree of destruction by earthquake are controlled by many factors, among which geological structures, foundation soil and topography are most important. The first is the geological structures. Particularly the location of rock structure on seismic fault controls the general trend of distribution of earthquake hazards. Generally speaking, the seismic intensity increases toward the seismic fault. The second is the propagation medium of earthquake waves, i.e. the nature of foundation soil. The Quaternary loose strata play a role of amplifying and intensifying the effect of the vibration caused by earthquake and may lead to the liquefaction and other invalidities of foundation soil. This factor is very important for the stability of bridge. For example, in the Tangshan earthquake region five railway bridges located in the pleistoseismic zone, but not directly on the fault displacement zone and possessing a solid foundation, had successfully withstood the test of seismic intensity of 10-11 degrees and had not been damaged seriously. On the contrary, some bridges, though located in the area of 7 to 8 degree seismic intensity and tens of km. away from the epicenter, had been seriously damaged because their foundation soil is loose and soft and subjected to liquefaction by tremors. In respect of the topographical factor, the representative case is that if the abutments of a bridge are built on slopes or isolated mounds, the bridge would be damaged by landslide. Due to the difference in the above factors, abnormal zone of low seismic intensity may occur in area of high seismic intensity, and vice versa. For instance, the district of Yutian County, Hebei Province is a "safety island" of low intensity in an area of high intensity.

The conditions for a "safety island" can be summarized as follows: 1) It is a relatively complete block with good stability and simple internal geological structures, and bounded on all sides by faults; 2) it has solid foundation soil. It would be best if the bedrocks is exposed or buried shallow; and 3) the river banks on it are stable with gentle slope.

For example, the Beijing-Qinhuangdao Railway crosses the Luanhe River and the Qinglong River and long bridge need to be erected on them. As shown in Fig. 6, this is an intersection area of the NNE-trending fault and the NW-trending fault. The original long bridge of the Beijing-Shanhaiguan Railway is located south of the intersection area, crossing the NNE-trending fault (Fig. 7). In 1976 when the Tangshan earthquake occurred, this fault was not a seismic one but was active due to dragging effect by the seismic fault, resulting in the comparatively serious damage to the Nos. 11-21 bridge's piers and the eastern abutment. The western abutment and Nos. 1-10 piers were only slightly damaged because they were located outside the dislocation zone of the active fault and were built directly on solid bedrocks. It is very important for the determination of bridge site in an active fracture zone to avoid direct crossing of active fault and to select a solid foundation. The new sites of No. II and III are not suitable due to the following fact that they are not only crossing the active faults directly, but located at a intersection of alternatively ascending and descending fault blocks during the neotectonic. That is the most
Fig. 6. Comprehensive analysis of structural stability of the long Luanhe River bridge.

1. NNE-trending active fault; 2. NW-trending fault; 3. Migration direction of channel; 4. Exposed area of bedrocks; 5. Historical epicenters; 6. Epicenters in recent years (large circles representing earthquakes of magnitude > 5); 7. Contour of surface vertical deformation in recent years; 8. Selected bridge site.

Of course, the problem for predicting the activities of faults and probably location for the formation of a lock structure. It is still worse that the strata of these two site's foundation are loose and more than 30 m thick, some of which belong to the potential liquifaction layers. Therefore, they are not suitable for building bridges. Site No. 1 is located north of the intersection and does not cross the active fault, and is on a relatively complete block with shallow and solid bedrocks which are favourable for the base of bridge. This site can be considered as a "safety island" and has been accepted for bridging.

There are long bridges along the Beijing-Shanhaiguan railway.

Fig. 7. Geological section of the site of the long Luanhe River bridge.
the location of potential earthquake
epicenters at present is still not ripe.
However, the comprehensive exploration
and analysis, based on the abundant
data of investigation and results of
the seismotectonical research, are
found to be helpful to guide the deter-
mination of railway bridge site and
will certainly be of significance for
the safety and stability of railways
engineering.

(1981, 8)

Reference

Ma Zongjin et al, "Structural Analysis
of Nine Strong Earthquakes in China in
Recent Years" Northwestern Transactions
of Seismology, No. 1, 1980.
SEISMICITY STUDIES OF SOME IMPORTANT DAMS IN INDIA

DES ETUDES DE SISMICITE DE QUELQUES BARRAGES IMPORTANTS EN INDE

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ABSTRACT
Special Seismological investigations have been undertaken in India around Pong, Pandoh, Bhakra, Salal, Dihang, Subansiri and Koyna dams by operating a network of high gain stations. Of these, a network of ten Seismological Observatories around Pong and Pandoh dams operating since 1963-64 has yielded interesting results to compare the seismicity during the pre-pounding period with that after filling the reservoirs.

In the Koyna region, the revaluation of epicentral parameters with the help of a velocity model deduced from the Deep Seismic Sounding explosion experiments has indicated a shallower focus events in the region. The orientation of fault from this data has been compared with the earlier studies.

Temporal variations in seismicity in the vicinity of Pong and Pandoh dams do not show any marked change after impounding of dams for events of magnitude 3 or more. The values of 'b' in Gutenberg-Richter's frequency-magnitude relationship do not show any significant variation prior to and after impounding of dams implying not much changes in the stress level in the region. A short term microearthquake survey in the vicinity of the proposed dam near Dihang and Subansiri river has indicated slightly higher level of seismic activity in the vicinity of Dihang dam. However detailed investigations are called for.

ABSTRACT
On a conduit des études seismologique spéculaux autour des barrages a Pong, Pandoh, Bhakra, Salal, Dihang, Subansiri, Koyna par l'operation d'un reseau des dix stations Seismologiques fonctionnant depuis 1963-64 autour les barrages a Pong et Pandoh a produit des resultats tres interessants pour faire la comparaison entre la valeur de seismicite avant et aprés remplissage de la barrage.

En la region de Koyna, en revaluant des parametres epicentrales a l'aide d'une modele velociet deducre des experiences "explosion profond de Sondage Seismique (Deep Seismic Sounding Experiments). On a trouve des indication (le'existence) d'un peu profond foyer devanement (Shallower focus events). On a comparle' orientation de la faille avec les anciens resultats. Pour les evenement de grandeur 3 ou plus, la variation temporelle de seismicite avant et aprés remplissage de la barrage n'est pas remarquable. Les valeurs de 'b' en la 'Gutenberg - Richter' frequency-grandeur expression ne montre pas grande variations pour des deux cas-avant de remplissage et aprés de remplissage de la barrage, en indiquant un peu de changement en niveau de contrainte de la region.

Un leve micro-seismique a couvert terme en proximit des barrages proposees près des fleuves Dihang et Subansiri a indique un peu plus grand niveau de seismicite près de barrage Dihang. Cependant, il faut faire plus des recherches en cet cas.
Introduction

In India a number of high dams are being constructed in the seismically active region of the Himalayas which is the source of many rivers. Of these Bhakra dam was the first to be constructed. More recently Pong and Panchgani dams have been constructed in Himachal Pradesh which are located within 100 km from the great Kangra earthquake of 1905. These are one of the few high dams in the world for which seismological data is available several years before the impounding of the dam by operating a close network of ten seismological stations. A short term microearthquake survey was also undertaken in the vicinity of Dihang and Subansiri rivers for assessing the seismic status of the region. The seismicity around Koyana dam in the Peninsula India still continues to be interesting. Eventhough the aftershocks of the damaging earthquake of Tenth December 1967 in Koyana have gradually decreased, occasional spurt of seismic activity during post monsoon months still causes concern and raises doubt whether the events are precursory to a more damaging earthquake in the region.

As is well known, there have been about a dozen cases in the world in which the seismic activity has shown a remarkable increase after the impounding of the dam. On the other hand, some studies have shown a marked decrease in the seismicity or its absence just after the filling of reservoir in active seismic region. Thus detailed seismological investigations are needed for dams located in widely different tectonics to understand the genesis of reservoir associated seismicity.

The object of this paper is, therefore, to examine the seismicity variations before and after impounding the Pong and Panchgani dams. The data has also helped in understanding the seismicity around Bhakra dam by making use of the additional data provided by an observatory located at the dam site. The seismic events of magnitude 4 and more around the Koyana dam have also been relocated by an accurate velocity model deduced from the controlled explosions. These have been compared and interpreted with reference to the results of earlier workers. The result of short term microearthquake survey around Dihang and Subansiri dams over the tributaries of the Brahmaputra river have also been presented in this paper.

2. Seismic Network

Figures 1, 2, 3 and 4 show the location of seismological stations around Pong, Panchgani, Bhakra, Dihang and Subansiri dams maintained by the India Meteorological Department. For earthquakes originating in the Koyana region, the seismological stations at Karad, Poona, Bombay and Goa are supplementing the information provided.

Fig. 1: Epicentral map around Pong and Panchgani Dams (Himachal Pradesh, India) region for the period 1965-1974.
Fig. 2, Epicentral map around Pong and Pandoh Dams (Himachal Pradesh, India) region for the period 1975 - 1980.

Fig. 3, Epicentral map of Bhakra Dam region.
by the station maintained by the Central Water and Power Research Station, Poona.

For monitoring seismic activity near Dihang and Subansiri rivers, a short term microearthquake survey for about one month was conducted. The instruments were located at Pasighat, Along, Zero and Gerigamich. It may be seen that due to non-availability of rock outcrop at these station full use of the high gain instruments could not be made.

Seismicity and tectonics

**Pong and Pandoh**

Pong and Pandoh dams are located in the vicinity of the main Himalayan boundary thrusts. This thrust is locally called Satlitta thrust and lies about 2.7 km down stream of the dam axis, dipping at an angle of 20 to 30° towards north. At this place, the upper Siwalik sand rock and clay shale beds have been folded into minor anticlines and synclines due to their thrusting over the sub-recent and recent Alluvial deposits. Kangra earthquake of 1950 had its epicentre about 60 km to the northeast of Pong dam site. During this great earthquake, the intensity experienced at the dam site was VIII M.J. scale. Krishnaswamy et al. (1970) have inferred that the earthquake was related to the downward extension of Satlitta thrust. Along this thrust, three periods of movement have been indicated. Due to the first movement, a cumulative vertical throw of about 1.5 km is anticipated to have taken place when the upper Siwalik sand rock and clay shale beds were thrust over the upper Siwalik Boulder Conglomerates probably during the middle Pleistocene. The second and the third movement may have taken place in the late pleistocene and recent times.

Studies of focal mechanism of earthquakes in the region indicates that the nature of faulting is predominately thrust and dip slip type. (Das Gupta et al 1981) which is in agreement with the tectonics of the region. Pressures are acting at right angles to the faults and are shallow dipping. The geotectonic setting near Bhakra dam is similar to that of Pong and Pandoh dams.

**Koyana dam**

The area of Koyana dam region is covered by Deccan Trap consisting of rocks of different strength. The main types are massive basalt vesicular and amygdalo-oid basalts, tuff breccia and red hole. Mapping by the Geological Survey of India, shows that in the Late Cretaceous topography there were marked variations in elevation and that the Deccan volcanics were erupted along many fissures. Eventhough major faults have been rarely observed in the Deccan Trap, geologists consider that the great scarp of the western Ghats is the results of major fault. Gohain (1969) assumed that the hot springs around the NNW-SSW zone of Kendara foot hill rises along fault zone within the basement. Seismological data (Randen and Chauhuny, 1966, Bhanger, 1972, Sykes, 1970) have suggested that Koyana earthquake might be due to fault running in NNW-SSW direction. Results of Deep Seismic Sounding however, give direction of faulting as NWW-SSW, associating seismicity of the Koyana area due to recent adjustment along the deep fault immediately west of Koyana (Kaila et al, 1980).

**Dihang and Subansiri dams**

These dams are located in northeast India where several earthquakes of damaging intensity have occurred in the past. The great earthquakes of 1897 (near Shillong) and 1950 (India-Tibet border) caused widespread damage in the region.

The Eastern Himalayas is characterised by large scale thrust movements. The main boundary fault near the foot hills of Himalayas, the Magha thrusts and the Dawki fault are the sources along which many earthquakes originate. Land-sat imagery of the region has shown several transverse lineaments along which earthquakes occasionally occur. The microearthquake survey in the region was taken towards the end of 1981.

**Earthquake parameters and epicentral maps**

(a) **Pong, Pandoh and Bhakra dams**

The epicentral parameters of earthquakes in the region were determined using the velocity model (Kamble et al 1974) given below

<table>
<thead>
<tr>
<th>P-wave velocity (km/sec)</th>
<th>Depth of the top of the layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.72</td>
<td>0.00</td>
</tr>
<tr>
<td>6.66</td>
<td>24.00</td>
</tr>
<tr>
<td>8.22</td>
<td>45.00</td>
</tr>
</tbody>
</table>

The earthquake parameters till 1978 were determined using manual methods. From 1979, the U.S.G.S. revised programme HYPO T1 was used for improving the accuracy in the determination by the two methods which were broadly in agreement. It may be mentioned that the magnification of the Pong observatory located near the dam had
to be reduced from 50K to 5K in the year
1974 after impounding the dam due to the
generation of microseism on the records.
This has set a limitation in the compar-
sion of seismic activity before and after
impounding the dam from a single station.
However, the detection capability of the
region remained unaffected for earth-
quakes of magnitude 2 or more which could
be easily recorded by the other stations
in the grid.

Figures 1 and 2 show the epicentral
map of the region for the period 1965 to
1974 and the other for 1975 to 1980. This
has enabled us to compare the seismicity
variations prior to impounding and after
impounding of Pong dam in May 1974. Such
a comparison however could not be done
for the Pandoh dam as only two years have
passed after its impounding.

The velocity model for Dihang-
Subansiri region used for epicentral
determination was as follows:

<table>
<thead>
<tr>
<th>P-wave velocity (km/Sec)</th>
<th>Depth of the top layer (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.95</td>
<td>0.00</td>
</tr>
<tr>
<td>5.58</td>
<td>2.00</td>
</tr>
<tr>
<td>6.55</td>
<td>24.80</td>
</tr>
<tr>
<td>7.91</td>
<td>46.30</td>
</tr>
</tbody>
</table>

Figure 4 shows the events detected in the region during the short term microearthquake survey.

In the Koyana region, Chaudhury et
al. (1982) have determined the following
velocity model from the two profiles of the Deep Seismic Sounding experiments in the Koyana region.

<table>
<thead>
<tr>
<th>Velocity (km/sec)</th>
<th>Depth (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.60</td>
<td>0.00</td>
</tr>
<tr>
<td>5.32</td>
<td>1.20</td>
</tr>
<tr>
<td>6.21</td>
<td>17.30</td>
</tr>
<tr>
<td>8.23</td>
<td>36.30</td>
</tr>
</tbody>
</table>

An additional layer of 1.2 km thick-
ness with P-wave velocity as which corre-
spond to the average thickness of the
Deccan Trap in the region was also included for epicentral determination.

Figure 5 shows the epicentral map of Koyana region for earthquakes of magni-
tude equal to a greater than 4 during the period December 67 through December 73.

Results and Discussion

(a) Pong and Pandoh Dams

It would be noticed that the loca-
tion of epicentres of earthquakes in the
region are by and large to the north-
eastern side of the dams implying their association with the main boundary faults. The focal depths of the events vary from
5 to 30 km, with relatively deeper events
located away from the faults justifying
the northeasterly dip of the geological
faults. Slight increase in the number of

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Fig. 4. Epicentral map of Micro-earthquake survey of Dihang-Subansiri Dams (Assam, India) region.
earthquakes after filling the reservoir is noted from figures 6 and 7. However, there is no significant change in the number of events recorded within 25 km radius at Sundernagar observatory close to Panoj dam. Similar relative comparison is however not possible due to reduction in the magnification of the instrument at Panoj after the impounding of the dam. It is nevertheless noticed that there is no significant change in the number of events of magnitude 3 or more on Richter Scale at either of the two dams before and after the filling of the reservoir.

It is interesting to note that the cluster of earthquakes towards northeast lies in Kangra valley where the great Kangra earthquake of 1905 occurred. The region is still seismically very active where earthquakes of moderate intensity followed by aftershocks occurred in 1963 and 1976.

Comparison of ‘b’ value during the pre and post, impounding period (figures 8 and 9) has shown clearly that there has not been significant change in the stress conditions in the region after the impounding of the dams (Safa, 1970, 1976). Even though absolute value of ‘b’ may change with a large population sample (Hsu, 1971).

In similar tectonic framework, Kangala dam in Pakistan, increases in local earthquakes, also could not be established due to changes in detection capabilities of local recording network (Adams and Ahmad, 1969). In the case of Tarbela reservoir in Pakistan also located in active fault zone did not indicate any correlation with water level. Instead decrease in seismicity after loading of reservoir was temporarily indicated (Jacob et al., 1975). It was inferred that in tectonic environments where the maximum principal stress is horizontal and the tectonics system is compressive, a vertical surface load can move the crustal stress away from a Navier-Coulomb failure criterion, implying, temporary decrease of seismicity. In the case of Bhakra also, no significant increase in seismic activity could be observed (Chauhun and Srivastava, 1978). It is well known that earthquake occurrence associated with reservoir implies the presence of tectonic stress. However, the permeability of rocks and the nature of faulting appear to play a vital role in triggering the earthquakes. Since the order of stresses induced by the water load of the reservoir is relatively much smaller than that needed to cause tectonic earthquakes, the sedimentary rocks near Panoj and Panoj dam are essentially different as compared to deccan traps where Koyne earthquake of 1967 originated, the differences in the pattern of induced seismicity are not unexpected.
Fig. 6, Seismic activity near Pong Dam.

Fig. 7, Seismic activity near Parnoh Dam.
Fig. 8. Frequency Magnitude Plot for earthquake within 50 km of Pong Dam.

Fig. 9. Frequency - Magnitude plot for earthquakes within 50 km of Pandoh Dam.
Bhakra Dam

In the case of dam, the events monitored in the region do not indicate any direct relationship with the reservoir. These events have, however, indicated two lineaments (as shown by dotted line) roughly at right angles in figure 3 to the main bound of fault.

Koyna Dam

Figure 5 shows the earthquakes located in the Koyna region using the new velocity model as mentioned earlier. Most of the events detected had their fold in the earth's crust. The focal depth of the main earthquake of 30th December 1967 was found to be 2 km as compared to 8 km reported earlier (Tandon and Chaudhury, 1968). This event appears to be well located using Hypo 71 with RMS value as 0.31 and error of 4.83 km in focal depth.

It may be seen that the events of magnitude 4 and above are oriented broadly along north-northeast to south southwest. The trend is more or less similar to the results of Tandon & Chaudhury (1968). Although the number of events is rather small, a few deeper events have been located towards the east. In order to draw firm inference regarding the more details of the fault in the region, it would be worthwhile to determine the epicentral parameters of large number of smaller events in the region. Deployment of a high gain microearthquake seismograph may resolve the problem.

Conclusions

(1) Location of epicentres around Pong and Pandoh dam region are by and large to the Northeast side of the main boundary fault indicating a northeasterly dip. This inference gets strength because even though all the earthquakes are of crustal origin a slightly deeper ones are generally located away from the faults. The result is in conformity with the known faults plane solutions in the region. Seismic activity does not appear to show a correlation with water level of the reservoir of both the dams. These appears to be a slight increase in the number of earthquakes after the filling of the reservoir. However, there is no change in number so far as events of magnitude 3 or more on Richter Scale in concerned. There is no change in the value before and after impounding of reservoir. Limitation of the lesser population density has however to be kept in view.

(2) The seismicity around Bhakra dam also does not indicate any increase.

(3) Epicentre of earthquakes of magnitude equal to or more than 4 on the Richter Scale in the Koyna region arrange themselves broadly in NNE to SSW direction. Comparatively deeper events locate themselves towards the east direction.

References

ABSTRACT

It is necessary to investigate vast areas while planning for Hydroelectric projects. The recent advancement in technology provides Geophysical methods for exploring large areas within a short duration. Seismic refraction is one such method, subsurface data interpreted from which can be used for planning and obtaining various parameters needed for design purposes.

The present field investigation comprises of seismic refraction survey, in-situ rock testing and drilling bore holes at critical locations to know the ground truth. The results of all these tests have been presented. An interpretation of the results has been done to locate various geological contacts, unconformities, etc.; and based upon these, the appropriate locations for various structures of the project have been decided. In addition, various subsurface horizons have been identified on the basis of seismic wave velocity contrast and are compared with the log of bore holes.

ABSTRAIT

Il est nécessaire d'investiguer une vaste terraine vide quand on dessine des projets Hydroélectriques. Le nouveau avancement dans la technologie fournit la méthode géophysique pour l'exploration de la large surface dans une courte durée. Seismic refraction est l'une de ces méthodes, donne sons-surface interprétée laquelle peut-être utilisé pour dessiner et pour obtenir différents paramètres qui ont besoin pour dessin du projet.

Maintenant, l'investigation du champ comporte l'inspection de la seismicque.
refraction, une immense expériences des rochers et perçant de sondage a les locations
critical du barrage pour savoir la vérité du fond. Les résultats de toutes ces expéri-
ments ont été présentées. Une interprétation de ces résultats a été fait pour la location
de différentes contacts géologiques, inconfirmés, etc., et base sur celles-ci, l'
etablissement convenable de plusieurs structures du projet ont été décidées. En
addition, différents horizons de sons surface ont été identifié sur la base de la
contrasté de vitesse d'ondes de seismique et sont comparées avec la bocage de trou de
sondes.

1. INTRODUCTION

The Project envisages utilization of waters of a major river for the generation of
power in the north western part of India.

The project layout includes a 125 m high concrete arch gravity dam, 6.5 km long 9.5 m diameter concrete lined power
tunnel on the right bank of the river, a 25 m dia., 103 m high surge shaft leading to a vertical pressure shaft 8.5 m in
diameter and an underground power house, housing three generating units of 180 MW each. The tailrace tunnel after crossing
under the main river, discharges into the river. 1.75 km downstream of the confluence of the main river with one of its tributary
The tailrace system comprises of 1.77 km, long tunnel, 9.5 m dia., a 0.56 km long cut and cover section and a 95.0 m long open channel. The concrete arch
gravity dam is proposed with its top level at El. 765 m with its deepest
foundation at El. 610 m.

2. GEOLOGY

2.1 REGIONAL GEOLOGY

The project is located in the lithotectonic province of the lesser
Himalayas of the Himalayan belt. The project area is occupied by a variety
of rock types comprising granite gneiss, granite, phyllitic quartzite slate, limstone,
carbonaceous phyllite, quartzite and volcanics ranging in age from precambrian
to upper tertiary. The entire sequence is highly folded and faulted. Several major
regional thrusts are exposed in the project area. Those are the Jutogh thrust
on the east, inside the reservoir about 1.5 km upstream of the proposed dam
axis and is normal to the river course and the shall thrust (phyllite/volcanic contact)
which cuts across the power tunnel, 75 m upstream of the proposed surge shaft.
The main boundary fault separating the Siwaliks from the Murrees has been
mapped on the South West of the Project.

The river passing through the lesser
Himalayas has carved out a narrow
(though not the narrowest in the vicinity)
assymetrical V shaped valley and its
flow from NW to SE is generally turbulent
The majority of the cross drainages are aligned parallel to the regional strike direction of foliation jointing.

2.2 Geological features at the Dam Site

The bed rock at the dam site consists of variable bands of grey to dark grey quartzitic and slaty phyllites belonging to the Dhundiar/Baranpur formations. The weathering of the phyllites vary from 2 to 6m. The foliation is trending in N 20° W-S 20°E to N 10°E-S 10°W and dips in the order of 45°-65° in easterly or upstream direction. The phyllites were subjected to excessive tectonic crustal activities which eventually caused tight folding, shearing and jointing. Recent decompression action in the carving out process of the Canyon resulted in the formation of exfoliation cracks fifty or more meters into the abutments. These cracks are generally parallel to the topography. The left bank slopes at 50° and the right bank slopes 60° at the dam site.

Exfoliation planes are tight and are often reinforced with quartz and calcareous filled veinlets. Shear fractures up to 30 cm wide were located on both the banks, most of them are clay filled and generally impervious. Joints are usually tight but some are open. Exfoliation cracks are prominent on the right bank but scarce and isolated on the left bank, most of the exfoliation cracks are only partially clay filled, open and vary in width from a few millimeters to 10 cm. One exfoliation crack which is 40 m. inside the right bank drift tunnel at El. 701 m is clean and open, but it horsetails into a series of smaller clay filled cracks in the lower drift at El. 672 m. The maximum depth of alluvium in the river bed portion is 18 m.

The major zone of dislocation (the Jutogh thrust, 1.5 km, upstream of the dam axis) is a tectonic boundary between overlying granitic gneiss and underlying jet black carbonaceous phyllite. Along this tectonic boundary, phyllites have been overridden by granitic gneiss. Thickness of Jutogh thrust zone varies from 200-500 m. No large faults have been detected below the river bed. Two narrow shear zones have however been located.

The project is situated in a highly seismic area and falls in the lithotectonic province of the lesser Himalayas and falls within zone V of the seismic zoning map of India. The project is located 45 km. from the epicentre of earthquakes, of magnitude 6 (1945 and 1947) and 60 km. from the earthquake of magnitude 8 (1905). The detailed studies of up-to-date information on the tectonic models of the Himalayas and past history of earthquakes in the project area were made and the peak ground acceleration has been fixed as 0.2 g.

3. Geophysical Surveys

VIII.231
The Seismic refraction method was used throughout at the project site for Geophysical investigation work.

The seismic survey was conducted both on land and over water. The areas surveyed included river bottom, dam abutments, intake and diversion area, parts of the power tunnel, power house and tailrace. In all 61 seismic lines covering a length of 11.2 km. were investigated. In the dam area, 13 seismic lines were completed. In this paper, the seismic refraction survey done at dam site has only been covered. Thirteen, Nx holes totalling 1100m. of drilling were done at the dam site. Water pressure tests at 3m and 5m intervals and grouting tests were done in these bore holes.

The seismic surveys were initially conducted to define any geological anomalies, so that boreholes could be better located. The bore holes results were in turn used to calibrate and interpret seismic refraction test data.

Geophysical surveys were mainly used for
(a) Locating the depth of overburden
(b) Delineating the bedrock profile.
(c) Finding the depth of alluvium in the river bed at and near the dam axis.
(d) Finding the zone of weathering.
(e) Finding out the depth of stripping required for dam foundations.
(f) Finding out the depth of sand layer in borrow area for construction materials.
(g) Finding out the Rock Mass Quality.

3.1 Equipment

12 Channel Seismograph with amplification and recording system was used for seismic refraction survey.

3. Procedure

Ground vibrations due to compressional waves caused by an elastic surface shock were produced by the use of explosives and were picked up by Geophones and transferred into electrical signals which were transmitted and amplified by each of the 12 channel of the recording system. A time break corresponding to the instant of impact was recorded on the 13th trace eliminating delay problems caused by electro-mechanical devices. The timing lines at 2 milliseconds intervals on the recording paper enabled timing of the first compressional wave arrival to be made with an accuracy of \( \pm 0.2 \) milliseconds. Charges upto 0.7 Kilogram were used. The charges were placed on the surface to expedite the work.

The gain of each amplifier was automatically adjusted by Automatic Noise Suppression(ANS) circuit to reduce the local noise level. Because of the high noise level mainly due to rain and rapids, the quality of recording fluctuated from poor to excellent. Besides, due to very damp and hot
climate many records were damaged. Such blasts were repeated to obtain better results.

Finally, each and every record was identified by survey data, seismic line no., location, position and shot point number, and quantity of explosives and depth of burial of the charge.

3.3 Investigations on Land

A seismic profile generally consisted of 12 Geophones equally spaced which were in turn connected to the recording system through a 26 conductor cable. The spacing used were varied from 2.5 m in shallow over burden area to 7.5 m and 10 m in deep over burden area. A seismic line consisted of many profiles of 12 Geophone spacing each. Two adjacent profiles were overlapped by two geophones in order to tie the profiles together allowing for continuous profiling. For each profile, at least 5 shot points were used. Two shot points were located at each end of the line at a distance equal to the spacing of geophones. Two other shot points were located at a distance generally equal to or greater than the length of the profile. The distance of those far off shot points were increased if the over burden was quite deep and it was checked in the field that the first arrivals of P-wave were passing through the bed rock. Another shot point was located between geophones No. 6 & 7 i.e. in the middle of profile to find out the com-

pressive wave velocity in the overburden.

The above mentioned method of minimum 5 blasts enabled to do continuous profiling using a double control (direct and reverse shots) of the refracting horizons. The direct and reverse shots were necessary to use Hawkin's method for detailed interpretation.

3.4 Investigations across the River

The standard hydrophone technique was tried to get seismic profile of river cross-section. However, this method did not succeed due to very swift current (10 m/s) in the River. A special alternative technique was used instead. This technique provides almost the same information but is more cumbersome and takes longer time to execute.

In this technique, geophones were located on each bank where the shot points would have been positioned in the standard technique. Similarly, the shot points were located on the river surface where the hydrophones would have been situated. For each shot, the first arrival of P-wave was recorded on each bank. A complete set of shots allowed for the constitution of Time-Distance graph with direct and reverse information which permitted to compute depth and velocities of bed rock.

3.5 Interpretation of data
The data collected was interpreted by Hawkins's method and the calculations were checked by critical distance method. The timing of the first arrivals were checked/rechecked during the interpretation. Corrections were applied to take care of weathering and terrain topography. The calculations of seismic velocity and the depth of overburden, etc were calculated at every geophone point i.e. at every 5m or less.

3.5.1 Hawkins's Method

In considering the engineering applications of the seismic method, it should be appreciated that considerable irregularity is present in surface elevations, refractor profiles and in the velocity, distribution of the near surface layers. This irregularity introduces problems in some interpretation methods. The time terms approach provided by Hawkins's method is simple and reliable for handling the irregular conditions. In this method the detailed analysis based on the isolation of time-terms at individual shot points and recording (geophone) positions were made and depths to refractors were calculated from these time terms yielding a detailed depth profile. A pocket programmable calculator was used for doing all the calculations at the field. All the thicknesses or depths were calculated normal to the refractors and corrections were then applied for terrain topography, etc.

3.6 FIELD INVESTIGATIONS

The seismic refraction survey profiles were laid at the dam site both on land and across the river. The location of the seismic lines designated as SL 1, SL 2, etc. are indicated in Fig. 1. The details of the results obtained from 13 seismic lines have been summarised in Table 1. The seismic lines done across the river have been designated with suffix (R). Fig. 1 also shows the location of drill holes which have been designated as DDH 1, DDH 2, etc; The seismic bed rock velocities at different points on the seismic lines have also been marked in Fig. 1. The bed rock and overburden profiles were found out for each line at every 5 metres. Fig. 2 and Fig. 3 show the typical sections of lines across the river and on the land.

3.7 DISCUSSION OF THE RESULTS

The old fluviatile terrace overlooking the dam axis on the left abutment which extends from Elevation 780m to 820 m was probed by four seismic lines SL-1-2, SL-1-3, SL-1-4 & SL-1-5 to know the thickness of terrace deposit and to find out the bedrock profile. The results showed that the bedrock depth varied between 8 to 25 m and the seismic velocity of the bedrock varied from 2500-4500 m/s.

The seismic line SL-2 -R indicated a low velocity zone of 2300 m/s on the right bank which was later proved by grout test in drill hole No. DDH-1 which was
<table>
<thead>
<tr>
<th>Line No.</th>
<th>Location</th>
<th>Overburden</th>
<th>First Layer</th>
<th>Bedrock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Velocity m/sec</td>
<td>Depth m</td>
<td>Velocity m/sec</td>
</tr>
<tr>
<td>SL-1-R</td>
<td>U/S of dam axis across the river</td>
<td>1500</td>
<td>1.6-8.0</td>
<td>-</td>
</tr>
<tr>
<td>SL-2-R</td>
<td>D/S of dam axis across the river</td>
<td>1500</td>
<td>0-11.9</td>
<td>-</td>
</tr>
<tr>
<td>SL-3-R</td>
<td>D/S of dam axis, across the river</td>
<td>1500</td>
<td>0-14.6</td>
<td>-</td>
</tr>
<tr>
<td>SL-4-R</td>
<td>At the Dam axis partly on land &amp; partly across the river</td>
<td>1500-1875</td>
<td>5-15.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1825-1875</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SL-5-R</td>
<td>Toe of the dam, across the river</td>
<td>1500</td>
<td>2-6.5</td>
<td>-</td>
</tr>
<tr>
<td>SL-1-2</td>
<td>Left bank, downstream of dam axis</td>
<td>400-425</td>
<td>1.9-6.5</td>
<td>700-1200</td>
</tr>
<tr>
<td>SL-1-3</td>
<td>Left bank, downstream of dam axis</td>
<td>500-550</td>
<td>0.3-4.5</td>
<td>1000-1100</td>
</tr>
<tr>
<td>SL-1-4</td>
<td>Left bank, downstream of dam axis</td>
<td>500-575</td>
<td>3.1-5.1</td>
<td>925-950</td>
</tr>
<tr>
<td>SL-1-5</td>
<td>Left bank, downstream of dam axis</td>
<td>525-625</td>
<td>1.0-5.5</td>
<td>1025-1050</td>
</tr>
<tr>
<td>SL-1-8</td>
<td>Right Bank Road Level</td>
<td>550-600</td>
<td>2.0-6.5</td>
<td>-</td>
</tr>
<tr>
<td>SL-1-9</td>
<td>Right Bank Road Level</td>
<td>450-750</td>
<td>1.5-6.5</td>
<td>-</td>
</tr>
<tr>
<td>SL-1-11</td>
<td>Left Bank perpendicular to dam axis</td>
<td>1000</td>
<td>11-16.5</td>
<td>-</td>
</tr>
<tr>
<td>SL-1-12</td>
<td>Across the dam axis at river level</td>
<td>1300</td>
<td>4.5-10</td>
<td>-</td>
</tr>
</tbody>
</table>
SL-1-(R)

FIG. 2. CROSS SECTION OF SEISMIC LINE ACROSS THE RIVER U/S OF DAM AXIS

LEGEND

SEISMIC WAVE [1500]
VELOCITY IN m/sec.
BED ROCK PROFILE

SCALE:

5 0 5 10 15 20 25 30 METRES
FIG. 3. CROSS SECTION OF SEISMIC LINE ACROSS RAVI RIVER AT PROPOSED DAM AXIS

LEGEND
- SEISMIC WAVE VELOCITY IN m/sec.
- BED ROCK PROFILE

SCALE:
6.5 13.0 19.5 32.5 METRES
nearest to this seismic line. The grout intake was of the order of 7000 kg.

The Seismic line SL-3-R which was near to Drill holes No. DDH 5,5A, 6&8 show the bedrock at Elevation ranging from 625m to 645m. The Elevation of bedrock obtained from the seismic survey ranged from 625m to 640m.

The seismic line SL-4-R was located on the dam axis and showed the bedrock at Elevation 628m. The bedrock level was revealed by drill hole No. DDH 10 at El. 625m.

The lines done across the river indicated that the seismic wave velocity were in the range of 3000-4500m/sec at the various cross sections at the dam site. There was a weak zone of velocity, 1900 metres/sec. on line SL-1(R). The lines done on land at the dam site, indicated, in general, velocities in the range of 3000-5000m/s. Weak zones in between were having velocity of 2000-2800 m/s. As the velocities are generally greater than 3000 m/s, corresponding to a good quality concrete with a Young's modulus of $1.55 \times 10^5$ kg/cm$^2$, it is concluded that generally the rock is of good quality and grouting will have to be done to improve the low velocity zones indicating greater jointing of rock at those places.

The depth to bed rock in the alluvial filled river bed was found to be between 7m to 16m, and those depths were found to be between 8.25m (Drill hole No. DDH-3) to 18m (Drill hole No. DDH-8), thus proving that the accuracy of interpreted rock profiles from seismic refraction survey. It is to be emphasized that the drilling of holes have been done much after the seismic refraction survey.

The drill holes drilled near the seismic lines show the bed rock level varying from 625-645.0 m and the seismic survey indicated in general the bed rock level varying from 625-640m. At particular places, the depths of bed rock as given by seismic survey and the drill hole data were in close agreement differences being within ±10%.

3.8 Young's Modulus of Rock ($E$)

The accurate determination of seismic velocities are important in engineering considerations since the velocity is controlled by the fundamental parameters of elastic strength and density.

The correlation of seismic wave velocities with the Young's Modulus of rock were done and the following values were suggested.

<table>
<thead>
<tr>
<th>Velocity (m/sec)</th>
<th>E (Young's Modulus) $E$ (kg/cm$^2 \times 10^5$)</th>
<th>$E$ (Young's Modulus) Static $E$ (kg/cm$^2 \times 10^5$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
<td>0.60194</td>
<td>0.516</td>
</tr>
<tr>
<td>3000</td>
<td>1.55457</td>
<td>1.296</td>
</tr>
<tr>
<td>4000</td>
<td>3.04767</td>
<td>2.540</td>
</tr>
<tr>
<td>5000</td>
<td>5.13734</td>
<td>4.280</td>
</tr>
<tr>
<td>6000</td>
<td>7.87087</td>
<td>6.560</td>
</tr>
</tbody>
</table>
4. Conclusions

Extensive investigations are absolutely essential and can curtail lot of uncertainties during construction. Any expenditure involved in investigations is paid manifold in ultimate savings.

The geological and geophysical investigations play a great role for efficient layout, design and overall economy of the project. Geophysical investigations can be made very rapidly and thus save lot of time. Seismic refraction method is undoubtedly the most suitable method for use in Civil Engineering Projects. This case history is an example of the effective use of seismic refraction method. The interpretation of this method is more detailed than from borehole logs alone. In conjunction with drill hole data and the general geological knowledge of the region, the geophysical surveys can predict the underground conditions economically. It is essential, however that the results of Geophysical investigations should be studied jointly by Geologists, Engineers, and Geophysicists and the interpretations should be made considering the limitations of Geophysical tests.

In this project, the bed rock computed from seismic refraction at the dam site work, were later proved to be accurate within + 10% of the actual depths found out by drilling. The seismic velocity in bedrock can be used as an index of rock quality.

5. ACKNOWLEDGEMENTS

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SIGNIFICANCE OF SEISMOTECTONICS IN RESERVOIR PROJECTS ON NARMADA RIVER

SIGNIFICATION DE SISMOTECTONIQUES AUX PORJETS RESERVOIRS SUR LA RIVIERE NARMADA

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Abstract

Four major gravity dams sited successively upstream of one another starting with 145 m high Navagam (Sardar Sarovar) at the terminal gorge on Deccan traps underlain by Bagh sedimentaries, 34 m high Maheshwar on Deccan traps and 44 m high Omkareshwar and 83 m high Narmada Sagar on Vindhya sedimentaries are planned to be constructed in the next decade across Narmada river. The fifth, 56 m high Bargi Dam Project under completion is founded on Deccan traps in the upper reaches of the river. An ENE-WSW mega lineament dividing the Indian Shield involving crust and mantle and comprising a zone of linear and arcuate faults and fractures traverses this valley pulsating from Precambrian to Pliocene times. This mega lineament delineated on the basis of geologic and allied evidences is associated with past earthquakes of medium magnitudes. Rectilinear depressions suspected as abandoned river courses aligned parallel to the lineament, low level microseismic activity and its temporal changes have further corroborated the fact that the entire Narmada valley is uniformly active, but the seismicity is moderate. However, considering the investment in the projects and public safety to life and property in the event of a disaster, to begin with a conservative seismic coefficient of 0.1 g to 0.125 g may be adopted for the design of dam structure. This may be modified by more careful assessment of the seismotectonic status of the valley, following modern trends in seismic evaluation. The reservoir induced seismicity being a prospect owing to impoundment of about 25 billion m$^3$ of water, the filling of the reservoirs has to be regulated with smaller increments (of water). The seismic activity, both pre and post impoundment, should be monitored by instrumentation.
Abstrait

Quatre barrages de gravité situés avec succès haut-courant de chacun autre commençant avec un hauteur de 145 Navagam (Sardar Sarovar) à la gorge terminale sur les trappes de Deccan étant au dessous de sédimentaires de Bagh, 34 m haut Maheshwar sur les trappes de Deccan et 44 m haut Omkareshwar et 83 m haut Narmada Sagar sur des sédimentaires Vindhyan sont projetés d'être construire à la décennie prochaine à travers la rivière Narmada. Le cinquième, 56 m haut projet de Barrage Bargi près d'être achevé est établi sur des trappes de Deccan aux atteintes élevées de la rivière. ENE-080 mégà linéament partageant la carapace indienne comportant croûte et manteau et comportant une zone des failles et des fracteurs linéaires et précises traversent cette vallée vibrant des époques pré cambrienne à piélistocène. Ce mégà linéament décrivant sur la base des évidences géologiques et alliés est associé avec des tremblements des magnitudes moyennes. Dépressions rectilignes soup connues comme des cours des rivières abandonnés alignées parallèle au linéament, l'activité microsismiques de niveau bassin et ses changements temporels ont corrobéré davantage le fait que la vallée entière de Narmada est uniformément active; mais la sismicité est modérée. Cependant, considérant l'inventaire des projets et sécurité de la vie et celle de la propriété publique au cas d'un désastre, à commencer avec un coefficient préservateur sismique de 0.1 g à 0.125 g peut être adopté pour le dessin de la structure de barrage. Cela, peut être modifié par une évaluation d'un état sismotectonique de la vallée, suivant des direction modernes dans l'évaluation sismique. La sismicité du reservoir induite étant une perspective à cause d'un endiguement de à peu près 25 billion m³ d'eau; le peuplement des réservoirs doit être réglé avec des augmentations (d'eau) réduites. L'activité sismique, dans tous les deux cas, avant et après l'endiguement, doit être contrôlée par instrumentation.

Introduction

Narmada River drains a variety of geological formations over a distance of more than 680 Km with peak floods of 80,973 m³/sec. The basin possesses 3000 MW of power potential and irrigation potential of 2,35 million hectares. The other development benefits are flood protection, navigation, fish culture and industrialisation.

For harnessing the above benefits four gravity dams are planned successively upstream of one another. The dams are the 145 m high Navagam (Sardar Sarovar) at the terminal gorge, 34 m high Maheshwar, 44 m high Omkareshwar and 83 m high Narmada Sagar. Apart from Navagam dam already under construction the 55 m Bargi dam project near Jabalpur is under completion. Geologists and Seismologists have recognised
that Narmada River flowing west and Son flowing east have been guided primarily by a major deep tectonic lineament associated with seismicity which has been variously designated as a fault system, rift valley, graben etc. The resulting reservoirs impounding 25 billion m³ of water, may be subject to earthquakes or in turn may induce seismicity. The dams and appurtenant structures for impounding such large reservoirs, must be designed to be safe against any disaster causing destruction and damage to life, especially human, and property. Presently the Geological Survey of India has undertaken Crust-mantle studies of the Son-Narmada and Tapti Lineament Zone called project CRUMANSOATA, envisaging integrated multidisciplinary studies comprising photogeological, geological, geophysical, geotechnical, seismic and other investigations for proper appraisal of this lineament. The above background has prompted the writing of this paper on seismic-tectonic status of the Narmada Valley for a precise evaluation of the seismic risk and seismic design of the civil engineering structures.

Narmada River

Narmada is one of the principal rivers in Central and Western India, descending from an elevation of 1057 m at Amarkantak in Maikal range and drains over a length of 680 km in a remarkably straight line, entering the Gulf of Cambay near Broach. From Amarkantak to Jabalpur towards north-west it has a sinuous course beyond which towards west it follows a straight trough, filled with alluvium extending upto Harda. From Harda towards west, the river flows over rocky terrain up to Garudeshwar. Further west, it enters into the alluvial plains finally joining the Gulf of Cambay. The Vindhyan hill ranges rise 300 m above valley on the north merging with Malwa Plateau (520-250 m). Sature Mahadeva and Maikal ranges flank the river on the south forming a series of scarped plateau between 600-900 m. In the upper reaches the rocky terrain is made up of structural hills trending ENE-WSW in Archaean terrain; residual hills and low-lying valleys in Gondwana areas, cuestas, scarp, hog back and shales-valley landforms in Vindhyan tracts and dissected plateau in Deccan trap country. The Narmada alluvial tracts consist of terraces, old meander scars and eroded piedmont fans. In the lower reaches the Deccan-Trap-terrain is formed of rugged linear ridges and valleys differing from characteristic step like topography of trap country. This feature is partly controlled by dykes and fractures in the area. The dendritic drainage is fairly mature but incised at places. It is controlled by lithology and structure.

Geology of the Narmada Valley

In its course, Narmada passes through geological formations ranging from the Archaean to the
recent. The straight course of the Narmada is attributed to faulting and the river is of the post-Deccan trap age. The sequence of the formations (Fig. 1) is given in table 1.

Narmada Fault System

The Narmada-Son Lineament has been a subject of geological conjecture. It has been identified as a fault zone, graben, rift or lineament on the basis of conspicuous ENE-WSW trending narrow geomorphic features right from the west coast of Gujarat to Chota Nagpur plateau in Bihar, dividing the Indian shield, with postulated extension on either direction which are not known. The Narmada lineament defined by the Narmada Valley is a part of the mega Narmada-Son-lineament. Therefore in this paper features pertaining to the former will be described in terms of the latter (Narmada-Son lineament). The Narmada lineament is identified by parallel system of several arcuate and linear faults, fractures and Swarm of dykes spread over a large area, geomorphic features, geological and geophysical evidences, Quaternary neotectonic movements characteristic of rejuvenation, thermal springs and associated centre of seismic activity. (Fig. 1) are elucidated below:

a. Geomorphic Features;

The Narmada river is flanked by Vindhyan-Kaimur and Satpura-Maikal hill ranges with long scarps, prominently aligned ENE-WSW. The WSW river course is remarkably straight over long distances; it flows through an entrenched valley in Jabalpur-Hoshangabad alluvial tract (Roy, 1981). There are two terraces at two levels, showing tilting and north flowing tributaries indicate offsetting (Bedi, 1973; Shenoj, 1977; Srinivasan et al, 1981) in the alluvial plains in Gujarat. In the alluvial tracts, river capture is a common feature in the valley.

These are typical features suggestive of structural and tectonic controls in the geomorphological evaluation of the river. On aerial photographs rectilinear depressions suspected to be abandoned course of the Narmada or paleovalleys have been detected near (within a distance of 10 km) Narmada Sagar Project (Rajurkar, 1981).

b. Geological Evidences

The Narmada-Son lineament divides the Indian Peninsula into a southern shield and a northern Vindhya Plateau which comprise contrasting geological environments.

<table>
<thead>
<tr>
<th>Southern Shield</th>
<th>Northern Vindhyan Plateau</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Greenstone granitic complex of Archaean age</td>
<td>Uncommon</td>
</tr>
<tr>
<td>2. Upper Proterozoic Volcanics are predominant</td>
<td>Uncommon</td>
</tr>
<tr>
<td>3. Gondwana Super Group and continental deposits of Jurassic are very common</td>
<td>Upper Proterozoic Vindhyan sediments are dominant</td>
</tr>
</tbody>
</table>

Magmatic emplacements trending
<table>
<thead>
<tr>
<th>Formation</th>
<th>Age</th>
<th>Dominant rocks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Recent alluvium</td>
<td>Quaternary-Pleistocene</td>
<td>Gravels and sands, often cemented by lime</td>
</tr>
<tr>
<td>Older alluvium</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gaj beds</td>
<td>Tertiary-Eocene-Miocene</td>
<td>Conglomerates, grits, ferruginous sandstones, limestones and clays</td>
</tr>
<tr>
<td>Ankaleshwar beds</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deccan Traps</td>
<td>Mesozoic-Tertiary-Cretaceous-Paleocene</td>
<td>Basalt flows, their differentiates and dolerite intrusives</td>
</tr>
<tr>
<td>Bagh beds and Lameta beds</td>
<td>Mesozoic-Jurassic</td>
<td>Sandstones, shales and limestones (sometimes fossiliferous)</td>
</tr>
<tr>
<td></td>
<td>Upper Cretaceous</td>
<td></td>
</tr>
<tr>
<td>Upper Gondwanas</td>
<td>Triassic-Jurassic</td>
<td>Sandstones, shales and conglomerates with minor coal seams</td>
</tr>
<tr>
<td>Lower Gondwanas</td>
<td>Upper Carboniferous-Permian-Lower Triassic</td>
<td></td>
</tr>
<tr>
<td>Vindhyans</td>
<td>Upper Proterozoic</td>
<td>Quartzitic sandstones, siltstones and shales; intrusive basic dykes</td>
</tr>
<tr>
<td>Aravallis</td>
<td>Middle Proterozoic</td>
<td>Schists, phyllites and quartzites and granite intrusions</td>
</tr>
<tr>
<td>Bijawars</td>
<td>Lower Proterozoic</td>
<td>Schists, gneisses, quartzite sandstones and limestones. Basic intrusives (?)</td>
</tr>
<tr>
<td>Archaeans</td>
<td>Archaeans</td>
<td>Undifferentiated granite gneisses, phyllite, marbles, schists etc.</td>
</tr>
</tbody>
</table>

Tab. 1: Sequence of Formations

ENE-WSW are distributed en echelon along the Narmada Valley. They cut across Vindhyans (Narmada Sagar and Omkareshwar dam sites), Bagh sedimentaries, and Deccan traps (Navagam dam site) and Deccan traps (Maheshwar dam site), thus indicating their occurrence over a wide time scale from Precambrian to post-Cretaceous times. Auden (1949a) observed that the dyke swarm occurring in great force attains a width of 100 km along Narmada and Tapti rivers. Their dominant ENE-WSW trend and extensive occurrence lends support to the view that these have been emplaced along the existing planes of weakness caused by a mega fault.

A sequence of thick Cretaceous Bagh sedimentaries of marine origin occur below the Narmada river bed with a linear disposition.

At the Navagam dam site and the downstream alternatives, a reverse fault trending ENE-WSW to
EW have been delineated from the subsurface exploration (Shenoi, 1971; Raju and Shenoi, 1975; Srinivasan et al 1981) over a distance of 2 km along the river channel. At Narmada Sagar dam site a number of ENE-WSW faults exhibiting lateral movements have been mapped. At Omkareshwar dam site an EW fault along the Narmada river (flowing westerly) has been detected in addition to an ENE-WSW trending sheared dolerite dyke.

c. Thermal Spring along Narmada Lineament:

A number of significant thermal springs (Table 2) at Babeha, Anhoni, Anhoni-Samoni, Budi, Navagam dam site, and heat flow anomaly at Ankaleshwar oil fields are located in the Narmada Valley. The Navagam-dam-site-spring located close to the main river-channel-fault discharged water at 32°C for over a period of 4 months (Srinivasan op cit).

<table>
<thead>
<tr>
<th>Name of Spring</th>
<th>District</th>
<th>Location in°</th>
<th>Temp in°C</th>
<th>Discharge in l pm.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Babeha</td>
<td>Mandla</td>
<td>22°44' 38° - 80°24&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anhoni</td>
<td>Chhindwara</td>
<td>22°35' 56° - 50° 78°36' 20°58°</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>Anhoni-Samoni</td>
<td>Hoshangabad</td>
<td>22°36' 45° - 5° 78°21' 100°</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Budi</td>
<td>Chhindwara</td>
<td>22°57' 31° - 78°04'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sardar Broach</td>
<td>Sarover dam site</td>
<td>21°48' 32° - 73°46'</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Tab.: 2 Hot Springs—Narmada Valley

Krishnaswamy (1977) associated the thermal springs and heat flow anomaly with the crust mantle relationship and seismotectonics of Narmada Graben. Ravishankar (1977) related the ruptured crust along the lineament with partial melting of the upper mantle and emplacement of plutonic complexes at shallow depths, which might have caused heat flow, thermal springs and gravity anomalies along the lineament.

d. Geophysical Evidences

Based on Paleo-magnetic studies on Deccan lavas, Pal and Bhinasankaram (1976) have suggested that the lineament is a wrench fault or step fault genetically associated with post Jurassic drift and Deccan volcanism. Kallasam (1976) located a gravity high along the lineament and interpreted two linear faults defining the same. A number of aeromagnetic anomalies have also been noticed along the lineament.

e. Neotectonic Evidences

Badi (op cit) located two terrace levels on either bank of the river in the alluvial plains downstream of Gurudeshwar. These terraces are slightly tilted. He observed non-cyclic deposition at the river mouth. Quaternary faults along the confluence of Main river with Narmada river near Tilakuwada (21°57' 73°36') (Srinivasan op cit) and similar evidences near Mangrol (Das Sharma, personal communication) are infallible pointers to the neotectonic activity in the

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lower reaches of the river. The Broach area indicates 80 m down-throw in the alluvium on the northern side of the Narmada Valley. Auden (1969) recognised pronounced faulting over a distance of 230 km between Hoshangabad and Jabalpur affecting the Deccan lavas and quaternary sediments. In this linear tract in the upper reaches, the trends of isopach lines of alluvial cover is suggestive of graben-faulting of low magnitude. The base of the alluvium is lower than present day sea level, because of entrenchment of the Narmada river on account of faulting with possible reactivation as late as Pliocenec and recent times (Roy op cit).

Narmada Valley Projects

For the integrated development of Narmada Valley 5 multipurpose reservoir projects, (Table 3) are being implemented commencing with Bargi in the upstream and Navagam dam at the terminal gorge. The benefits issuing from harnessing of the river including its tributaries will amount to installation of sizable generating capacity of 3000 MW of power and development of irrigation to the tune of 2.85 million hectares. Additional benefits are navigation, flood protection, fish culture, conservation of wild life and industrialisation.

Seismotectonics

a) Tectonics:

The inter-relationship between Narmada and Tapti River courses are influenced by tectonic lines according to Vredenburg (1906). Crookshank (1935) elaborated the concept of "Narmada Rift Valley" as a result of pronounced faulting at the end of Deccan trap period.

Studying the Deccan trap dyke swarm and remarkable linear disposition of Narmada, Auden. (1949b) was of the view that Narmada-Son Lineament is major zone of linear weakness in the crust parallel to the Archaean grain. He considered this to have influenced the deposition of Vindhyan and Gondwanas. Spate (1954) ascribed the E-W Narmada Valley to a rising at time of imposition of stress fields related to Himalayan orogeny. He cited in support of the long scarps of Vindhyan-Kaimur and Mahadeva-Satpura hills and the straight coast of Saurashtra. West (1962) considered it as a line of weakness from early geological times with relative movements north and south of this line. Qureshi (1964) supported this postulation with gravity data indicating that the Satpura region represents a "horst".

<table>
<thead>
<tr>
<th>Project Features</th>
<th>Geology of foundations</th>
<th>Main benefits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bargi Project near Jabalpur - a 55 m high earth-cum-masonry dam.</td>
<td>Deccan basalt flows, horizontal in disposition and 18 to 42 m thick overburden. Paleo-valley and red bole beds are the weak features.</td>
<td>2,63040 ha of irrigation and 27 mw power</td>
</tr>
<tr>
<td>Gross storage: 1.22 B m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Narmada Sagar Project - Punasa - a 81 m high straight concrete dam.</td>
<td>Shallow dipping Vindhyan quartzites interbedded with silt stones cut up by five steep ENE-WSW faults. Decomposed weathered seams at bedding contacts and rock slides on left abutment are adverse features.</td>
<td>1,32,428 ha of irrigation and 108 mw power generation</td>
</tr>
<tr>
<td>Gross storage: 12.22 B m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Omkereshwar Project - a 44 m high earth-cum-masonry dam near Mandhata Island.</td>
<td>Shallow dipping Vindhyan quartzites, flagstones and shales cut up by two faults along two channels of the river and one sheared dolerite dyke trending ENE-WSW, close to the abutment. Soft seams and shales in foundations are problematic.</td>
<td>1,32428 ha of irrigation and 108 mw power generation</td>
</tr>
<tr>
<td>Gross storage: 1.50 B m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maheshwar Project - a 34 m high central spillway with earthen flanks.</td>
<td>Horizontal Deccan basalt flows with ENE-WSW dyke intrusion. Red bole bed, clinkery zones along the contacts and deeply weathered basalt flow constitute the foundation problems.</td>
<td>108 mw power and regulation of water supply to downstream Navagam dam.</td>
</tr>
<tr>
<td>Gross storage: 0.49 B m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Navagam Project - a 145 m high concrete dam and underground power house.</td>
<td>Pile of horizontal Deccan basalt flows overlying Bagh sequence of sedimentaries - sandstones, shales and limestones cut up by an EW river channel fault and profusely emplaced, by a network of dolerite and trap dykes. Red bole, clinkery flow contacts, shear zones and faults constitute the features requiring treatment.</td>
<td>260,000 ha of irrigation and 350 mw power</td>
</tr>
<tr>
<td>Gross storage: 9.462 B m³</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table: 3: Project features, Geology and main benefits.

as repercussion to crustal disturbance in Himalayas. He later (1971) opined that it is a mantle lineament, tectonically active from Precambrian times guiding deposition of Vindhyan and
Gondwana sedimentation. Yellur (1968) thought that the ENE-WSW and E-W rift faults and sympathetic faults with successive down throws controlling the Narmada River System have resulted in step-like disposition of faults. Auden (1969) recognised pronounced faulting over 230 km between Hoshangabad and Jabalpur affecting Deccan lavas and Quaternary sediments. He considered that the faulting commenced in Pliocene and continued into Pleistocene resulting in alluvium in the valley extending below the sea level. Drilling for groundwater supported this idea. Pal and Bhimasankaram (1971) postulated that the Deccan trap volcanism, spread of the Indian ocean flows and drift of the Indian plateau and the Himalayan Orogeny were simultaneous activities.

Auden (1960) extended the fault into shelf areas in Arabian sea. The ONGC Tectonic Map of India depicts the possible extension of the Narmada Lineament across the Gulf of Cambay to the southern periphery of Saurashtra Peninsula Crawford (1978) called the Narmada-Son Lineament as a deep narrow fault in Indian peninsula extending to Africa, based on tectono-magmatic studies. This feature commenced in India in Precambrian times and extended towards Africa in more recent times.

According to the conceptual model of Ghosh (1976), the Narmada-Son Lineament represents an erosional post Deccan Lava valley formed at the crest of a domal upwarp with tension fractures and shallow depressions along the crest.

b) Seismicity:

The Narmada-Son valleys are associated with major earthquakes of magnitudes varying from 5 to 6.5 which have occurred from 1847 to 1970. The epicentres of these events are distributed all along the valley (Table 4).

The microseismic survey conducted by Roorkee University in 1980 for the Navagam dam project revealed that the seismic level is about 2 events per day, the magnitude of event is less than 3, (many events are between 0 and 1) and there is temporal change in the activity of the region.

After the installation of seismological observatories in Gujarat State (lower reaches of the river) the shocks picked up around the Navagam project within 130 km between 1973 and 1980 are 137 and the maximum recorded magnitude is 3. (Pancholi, 1991).

The Deep Seismic Soundings (DSS) between Indore and Khandua has indicated 3 seismic events of magnitude 4.07 to less than 3, (Srivastava, personal communication) of which two are in the Narmada Valley.

Gubin (1969) delineated the Narmada-Son Seismogenic zone based on multiple elements like past-earquake events, size and nature of
faults, Quaternary tectonic movements etc., and indicated that once in hundred years, stronger earthquakes may recur at the location of the earlier earthquake. The shocks are because of contrasting differential tectonic movements along large faults, exposed or concealed, in this zone of "marginal areas of peninsula of uplifting plateaus". He mentioned that these faults may cut through crust and upper mantle. Chandra (1977) held that unlike earthquakes related to plate tectonics demarcated by mid oceanic ridges, and transform faults, the earth shocks of Narmada belong to intra plate earthquake for which plate tectonics offer no solution.

The Narmada, in view of the structural, geomorphic and seismic features already discussed, is active undergoing rejuvenation throughout its length. The microseismic activity may show temporal changes in time and space along the lineament, but earthquakes of higher magnitudes up to 6.5 may occur in the valley anywhere between Broach at the river mouth and head waters near Amarkantak. The epicentral locations from Broach to Rewa, revealing the even geographical distribution, also support this idea. The assumption that edges of ancient shields may be more seismic than the interiors is less definite in the case of Indian Shield (Auden, 1975). According to the quantitative seismicity maps prepared by Kaila (1972), the Narmada graben is a zone of moderate seismicity, Seismic design parameters for the proposed dams

From the foregoing review of seismotectonics it can be summarised that Narmada fault and graben system involving crust and mantle, is proven to be seismically active in the upper reaches from Hoshangabad to Jabalpur and also in the lower reaches downstream of Navagam dam site. The intervening reaches where the four projects are contemplated also show sympathetic parallel faulting, fracturing and dyke intrusives along ENE-USW direction. The parallel paleo valleys (?) located near Narmada Sagar on the air photos are being checked on the ground. The Narmada trans-current fault may have major strike slip disposition. The background microseismic activity recorded during OSS traverses from Indore to Khandwa and seismological monitoring at Navagam project (Srinivasan, op cit, Pancholi, op cit) confirm that the entire valley is seismically active with temporal and spatial changes in the distribution of seismicity and possibility of earthquakes of magnitude upto 6.5. All the parallel fault lineaments forming part of the Narmada system may not be active. Auden (1973) opined that fault encountered in the Navagam dam does not represent the main lineament. The microseismic studies proved that there is no clustering of events along this fault indicating
Tab. 1: 4 Epicentres of Important Earthquakes.

<table>
<thead>
<tr>
<th>No.</th>
<th>Location</th>
<th>Date</th>
<th>Month</th>
<th>Year</th>
<th>Lat. N</th>
<th>Long. E</th>
<th>Depth</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Dumohphahari</td>
<td>27</td>
<td>5</td>
<td>1847</td>
<td>21.40</td>
<td>75.00</td>
<td>-</td>
<td>6.5</td>
</tr>
<tr>
<td>2.</td>
<td>Barwani</td>
<td>18</td>
<td>11</td>
<td>1963</td>
<td>22.00</td>
<td>75.00</td>
<td>-</td>
<td>5.5</td>
</tr>
<tr>
<td>3.</td>
<td>Rewa</td>
<td>2</td>
<td>6</td>
<td>1927</td>
<td>24.00</td>
<td>82.30</td>
<td>-</td>
<td>6.5</td>
</tr>
<tr>
<td>4.</td>
<td>Satpura</td>
<td>14</td>
<td>3</td>
<td>1938</td>
<td>21.60</td>
<td>75.00</td>
<td>-</td>
<td>6.3</td>
</tr>
<tr>
<td>5.</td>
<td>Balaghat</td>
<td>25</td>
<td>8</td>
<td>1957</td>
<td>22.00</td>
<td>80.00</td>
<td>-</td>
<td>5.5</td>
</tr>
<tr>
<td>6.</td>
<td>Mahadeo Hills</td>
<td>26</td>
<td>3</td>
<td>1969</td>
<td>22.6</td>
<td>71.1</td>
<td>-</td>
<td>4.2</td>
</tr>
<tr>
<td>7.</td>
<td>Broach</td>
<td>23</td>
<td>3</td>
<td>1970</td>
<td>21.60</td>
<td>72.96</td>
<td>8.0</td>
<td>5.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>1</td>
<td>1967</td>
<td>21.97</td>
<td>74.27</td>
<td>-</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24</td>
<td>8</td>
<td>1974</td>
<td>22.90</td>
<td>73.60</td>
<td>33.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>23</td>
<td>2</td>
<td>1975</td>
<td>23.20</td>
<td>78.00</td>
<td>33.0</td>
<td></td>
</tr>
</tbody>
</table>

It to be quiescent. At the Narmadasagar and Omkareshwar dam seats, faults parallel to ENE-WSW direction may be of this category—quiescent—but microseismic monitoring has to be carried out in these areas to confirm this.

In conjunction with Gubin's (op.cit) multiple element seismogenic zoning attempts should be made to evaluate seismotectonic environments. Parameters like the largest earthquake that can reasonably be expected to occur at a site, the periodicity of potentially damaging earthquakes and the ground motion characteristics, the character of deformations and their possible severity and the potential for induced seismicity due to impoundment and consequential damage, should be judged. Of primary importance is the study of active faults, viz. their geometry, extent, temporal history, and the correlation with past seismic events. Static and dynamic properties of site materials must also be determined.

Detailed seismotectonic studies, when completed for Navagam (Sardar Sarovar), Narmada Sagar, Omkareshwar, Maheshwar and Bargi Projects will ensure a comprehensive understanding of the Narmada-fault-lineament seismic-activity which will be significant in the design of the civil structure. The establishment of observatories at Navagam dam, Broach area, and Bargi are the beginning of this long term objective. Though the Narmada fault is a persistent zone of moderate seismicity, considering the huge investment in the dams, public safety and unimaginable loss to life and property in case of disaster a conservative seismic design incorporating 0.1 to 0.125 g may be adopted for the gravity structures. As more and more data from the studies of seismicity parameters accrue, a more realistic seismic coefficient will evolve. The im-
pounding of the valley by a number of high dam creating storage of 25 billion m$^3$ of water with high hydraulic heads may induce seismicity and trigger off earthquake shocks relieving the tectonic stress. Guha, (personal communication) indicated that statistically Sarawati, Koyna and Ukai reservoirs which are closer to the more seismically active peninsular margin are frequently visited by earthquakes. This assumption may be less definite according to Auden, (1975) and much less so far as the Narmada Valley. Regulating the filling of the reservoir in small increments may precipitate smaller shocks which can relieve locked up tectonic stresses from time to time and avert the precipitation of a large event. Monitoring of the seismic activity, by a network of seismographs, before and after the reservoir impoundment will be rewarding.

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SOME RESULTS OF COMPREHENSIVE GEOLOGICAL AND GEOPHYSICAL SURVEYS OF SEISMOTECTONICAL PROCESSES AT THE AREA OF INGOURI PROJECT

QUELQUES RESULTATS DES VUES GEOLOGIQUES ET GEOPHYSIQUES COMPREHENSIVES DES PROCESSUS SISMOTECTONIQUES A LA REGION DU PROJET INGOURI

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ABSTRACT

On the basis of detailed geological and geophysical surveys at the initial stage of filling of the Ingouri reservoir, there have been considered the results of studies of seismotectonic processes in the earth's crust on the area of Ingouri Project. Deformation phenomena have been revealed in the near-surface of the earth's crust in the headwater reach and in the zone of effect of the reservoir. These phenomena are associated with seismic activity. In the headwater reach of the reservoir deformation processes and seismic activity were confined to the zone of intersection of regional tectonic fractures - the Inghirah fault and faults of the Tkvarcheli flexure. Under the action of the reservoir a stress relief in the form of earthquakes of $M = 1.1-1.6$ at a low energy level took place in this zone. In the zone of the Tkvarcheli stepfold at a distance of 20 km from the Ingouri reservoir and its effect contributed to acceleration of development of deformation process and relieving of tectonic stresses which took place in the form of a powerful swarm of earthquakes of magnitude up to 4.4. The strongest earthquakes felt in the epicentral zone with macroseismic intensity 7 degrees on the MSK-64 scale. Geophysical and geodetic observations at the stage of Project completion and creation of a deep reservoir (270m) allow to expect new data about development of deformation and seismotectonic processes.

RÉSUMÉ

Dans le rapport, sont examinés les résultats d'études des processus sismoto-tectoniques dans l'écorce terrestre du site du barrage d'Ingouri, obtenus à base d'un complexe d'études géologo-geophysiques détaillées lors du remplissage initial de la retenue. Des déformations ont été révélées dans la partie de l'écorce, proche de la surface, de la zone aval de la retenue de la zone d'influence de celle-ci. Les processus de déformation dans la zone aval de la retenue correspondaient à la zone d'intersection des failles tectoniques régionales d'Inghirichi et des failles de la flexure de Tkvarcheli.

Dans cette zone, sous l'effet de la retenue, il y a eu une détente à un niveau énergétique bas, sous forme d'un séisme de $M = 1.1$ à 1.6. Au point de flexion de la flexure de Tkvarcheli, à 20 km de la retenue.
d'Ingouri, l'effet de cette dernière a contribué à accélérer le développement des déformations et la détection des tensions tectoniques qui s'est produite sous forme d'un essaim puissant de séismes d'une magnitude allant jusqu'à 4.4. Les secousses les plus fortes se sont produites dans la zone épizentrale, avec une intensité allant jusqu'à 7 sur l'échelle MSK-64. Les observations géophysiques et géodésiques qui continuent au stade d'achèvement du barrage et de remplissage d'une retenue profonde (270 m), nous laissent l'espoir d'obtenir de nouveaux résultats sur le développement de déformations et de processus sismotectoniques.

It is known that the task of comprehensive studies of seismotectonic processes or processes of preparation of seismic events is very urgent for hydropower engineering. This comes from the fact that a considerable part of modern large hydraulic structures is designed, constructed and operated in the areas with high level seismic activity. In the USSR these are Torkugul, Nurek, Rogun, Ingouri, Chirkey, Charvak and some other projects located in the zones of intensive seismotectonic processes where occurrence of earthquakes of intensity 8 degrees and more (on the MSK-64 scale) are possible.

It was found that construction and operation of hydraulic structures with deep and lengthy reservoirs influence on intensity and directivity of tectonic processes and very often are the cause of different deformation phenomena, such as large landslides and downfalls, significant deformations of the earth surface, strong local earthquakes, etc. These phenomena are a serious hazard to hydraulic structures under construction and in operation. Therefore these phenomena shall be studied, predicted and taken into account in the design in advance together with evaluation of seismic danger of the areas under construction.

A type program of investigation of seismic danger of construction sites of large hydroelectric schemes was worked out in the USSR about ten years ago (4).

During the past period in the process of realization of its main statements in hydroelectric stations in the Central Asia and in the Caucasus some experience was gained which allows to judge about effectiveness of various types of studies. On the basis of results obtained in the area of the Ingouri hydroelectric station the main results of comprehensive geological and geophysical studies of seismotectonic processes during the filling of the deep-water Ingouri reservoir are discussed below.

1. Tasks of Studies and Long-Term Observation System

The Ingouri reservoir is located in the zone of Alpine folding, in the region of joint of the southern slope of the Great Caucasus and Transcaucasian middle massif (Georgian block) (Fig. 1). The head portion of the reservoir is intersected by the regional Tkvarcheli flexure dividing the Abhazsko-Rachinskaya upheaval zone and the Central-Megrelskaya depression. The Tkvarcheli flexure in plan has a series of geniculations which are connected with a system of fractures in a crystalline basement. The zone of the reservoir covers also the Ingirish upthrow-strike-slip fault intersecting the Tkvarcheli flexure at about 2 km from the dam (3, 7, 10, 11). The mean level of seismic activity for the whole Western Caucasus is rather low ($A_{10} = 0.1$, $s = 0.5$). However the seismic regime is distinguished by sharp time fluctuations of seismic activity and by a strong space localization of seismically active zones.

According to evaluations made during the design period the seismotectonic situation of the Ingouri reservoir zone is determined by the following potential seismically-dangerous structures (Fig. 1) (3, 7, 10, 11):

1) Main Caucasian thrust $M_{max} = 6.75-7.0$, $A = 30$ km, $h = 20-30$ km, where $M_{max}$ - maximum possible magnitude of earthquakes for the given seismogenerating zone;
Fig. 1. Scheme of tectonic structure of the site of the Ingourti Project: Seismogenerating zones: 1 - Main Caucasian thrust; 2 - Kakuro-Uuskurkij upthrow - thrust - fault; 3 - zone of upthrow faults of the Middle part of the Abkhaz-Svanetskaya steppe; 4 - Ingirish upthrow - strike - slip fault; 5 - Tkvarcheli flexure. Triangles - seismic stations; rhombi - sites; Latin letters indicate geological age of rocks.

Δ - minimum distance from the dam to the epicenter; h - most probable depth of seismic focus.

2) Branch of the Main Caucasian thrust (Kakuro-Uuskurkij upthrow-thrust fault) - \( M_{\text{max}} = 6.5; \)

\( \Delta = 25-30 \text{ km}, h = 20 \text{ km}. \)

3) Fracture zone of the middle part of the Abkhaz-Svanetskaya bench, \( M_{\text{max}} = 5.5, \Delta = 10-15 \text{ km}, h = 5-10 \text{ km}. \)

The zone of a Tkvarcheli flexure between the Galidzga and Ingourti rivers was not considered as a seismically-dangerous zone for the Ingourti hydroelectric station because at the stage of designing any contemporary tectonic processes were not found and its seismic potential thereby was reduced down to \( M_{\text{max}} = 5.5. \) Nevertheless just in this zone the strongest earthquakes (\( M = 4.4 \)) occurred after the filling of the reservoir during the whole period of instrumental seismic observations in the zone of the Ingourti reservoir (from 1972), though magnitudes of these earthquakes did not reach the predicted values.

Complexity and variability of seismic situation around the Ingourti reservoir and particular importance of the Ingourti dam predetermined the necessity of organization of continuous control observations of deformation processes in the earth’s crust of the given area with the aim in view:
- to find and study the most deformable sections of the rock foundation where deformations and
fractures dangerous for the structure may occur;
- to determine the effect of reservoir creation and its operation on seismic regime of the reservoir zone;
- to study the reasons and dynamics of occurrence of induced "dam" earthquakes;
- to determine behaviour of the rock foundation and the dam by earthquakes action.

A permanent system of geophysical and topographic observations was set up within the radius of 50 km to solve the given questions. This system shall provide a timely reveal of different natural anomalies over the given area resulted from regional seismotectonic processes and on the basis of their revealing to ensure timely prediction of natural phenomena dangerous for the structure, i.e. local earthquakes, displacements along the most "dangerous" tectonic fractures, large landslides and downfalls in the zone of the reservoir, etc (5, 6, 14).

The basis of the given network are 12 permanent seismic stations (Fig. 1) ensuring reliable recording of earthquakes above energy class 7, K ≥ 7 (energy class of earthquakes K is related to magnitude M by equation K = 1.8M+4.0), over the whole area under observation and earthquakes K ≥ 5 (5) in the head portion of the reservoir. This system is supplemented by a system of regional observations: electrotelluric, gravimetric, precise multiple levelling of base profiles and triangulation as well.

At the section of the arch dam and in the headwater portion of the reservoir the observation network involves in a system of detailed instrumental geophysical observations: tiltmetric, and deformographic, seismic surveying, electrometric and seismometric (Fig. 2). Particularly thorough observations are being conducted on "survivability" of main tectonic fractures of the construction site: the Inqirish upthrow - strike - slip fault and its branch Right-bank fracture revealed in the right-bank abutment of the dam.

In this case the principle aim of observations is to check up the supposition of the design period obtained by preconstruction geological engineering surveys and preliminary designing. The design prerequisite supposes that during the period of construction and operation of the dam and the reservoir probability of differential movements along the Right-bank tectonic "fault" in the near-dam surface zone consolidated by engineering means provided by the design is very low and their value cannot exceed a few centimeters.

In the zone of this fault observation points equipped with Ostrovsky's seismotiltmeters, hydrostatic (water-tube) tiltmeters and quartz deformographs which operate in conjunction with the system of three-dimensional geophysical measurements - seismic survey, electrometric and ultrasonic.

Processes in the zone of intersection of the Inqirish fault with the Tkwalkeli flexure in the head portion of the reservoir are investigated now with seismic observations on the underwater geophysical profile stretching upstream from the dam (Fig. 2).

2. Main Results of Long-Term Observations

By the moment of the initial stage of reservoir filling up to 105 m (April 1976) substantial information had been gathered which allows to reveal particularities of development of different geophysical "background" fields in the rock foundation of the dam and in the floor of the reservoir.

According to the observational data for the period preceding the filling of the reservoir (up to April 1978) seismotectonic situation in the area under consideration was relatively stabilized. In 1974-1975 within a radius of 30 km the level of seismic activity was A_{20} = 0.15-0.4 and in 1973, 1976, 1978 A_{10} = 0. Before the filling of the reservoir most of earthquakes occurred within the folding zone of the Great Caucasian southern slope. In 1977, i.e. the year before the filling of the reservoir, 6 earthquakes K = 6-7 occurred of which 4 earthquakes took place at a distance of 10 km from the site. During the period of initial filling (up to 50-60m) 6 earthquakes,
Fig. 2. System of points of long-term geophysical observations at the site of the Ingouri arch dam: 1 - Ingirish fault; 2 - boundary of marginal folding dislocation; 3 - tectonic fractures; 4 - large fractures; 5 - reservoir boundaries; 6 - isolines of longitudinal wave velocity - km/sec; 7 - Ingouri arch dam; 8 - area of long-term geophysical studies; 9 - permanent geophysical profile; 10 - points of engineering - seismometric observations; 11 - seismic stations; 12 - points of tiltmetering; 13 - points of ultrasonic observations; 14 - points of observations on underwater profile; 15 - points of electrometric observations; 16 - bore hole of seismic sounding; 17 - points of blasts.

K = 5-7, took place at 13-27 km from the dam site (2, 5).

Tiltmeters and defrommeters together with the instruments of the geophysical network showed that within the zones of tectonic fractures rather low variable deformations were observed which resulted from different surface factors and had no connections with deep tectonic processes. The main reasons of variations of stress-strained state of the considered sections of the earth's crust before the filling of the reservoir were various technogenous factors: mining work, grouting and concreting, seasonal temperature variations and variations of water content in the massif with the zones of observed tectonic fractures as well.

In April 1978 the reservoir was filled during a week up to 100 m. With a rise in the reservoir water level the general character of the stress-strained state in the considered section of the earth's crust somewhat changed. This resulted in certain changes in the readings of the observation system within the zones of tectonic fractures and in drastic changes in seismic conditions.
In the zones of tectonic fractures deformation processes appeared highly active with a rise in the reservoir level. At first it occurred in the Right-bank fault and then as the water level approaches the Ingirish fault zone the activity of these processes was observed at zone of intersections of regional tectonic fractures. In this zone variations of measured geophysical parameters increased substantially. As it takes place, a distinct differentiation of intensity of observed variations took place in accordance with the confinement of the surveyed area to this or that geostuctural block. This fact attests that in the zone of intersection of regional faults during the filling of the reservoir deformation processes within different structural blocks displayed their specific peculiarities and consequently the upper section of this zone at the least is consolidated rather poorly.

The most substantial changes were found for the processes taking place in the opposite slopes of the Tkvarcheli flexure composed of rocks with different permeability. In fissured and well-permeable carbonate rocks of the southern slope of the flexure deformation processes are more intensive and complicated than in the northern one which is represented by a practically impermeable stratum of volcanic deposits of the Middle Jurassic age. Contrast character of processes in the slopes of the flexure results probably in breaking deformations in the zone of fracture which in its turn worse propagation of elastic waves through this zone and forced up the reflecting property of its boundaries and in drastic changes of readings of the geodetic observations network as well. Judging by the available data deformation processes for the past two years propagated to the depth of about 300-400 m.

Within the zone of the Right-bank fault deformation processes in different structural blocks developed rather uniformly if only with some time shift. It shows itself most vividly at the initial stages of reservoir filling (1).

In the first years after the filling of the reservoir the seismic situation around the Ingouri dam changed to the unexpected way. A month later the filling of the reservoir, when the water level was unvariable and as high as 100 m, in the investigation area 10 earthquakes with $K = 6-7$ occurred, of which 7 earthquakes were located at about 10 km from the dam and nearer. Most of these earthquakes were confined to the zone of intersection of the Ingirish and Tkvarcheli faults besides 8 earthquakes occurred during 8 days. The May swarm of earthquakes clustered in time and space may be considered as the "dam" type and illustrates vividly an example of animation of potentially selfgenerating structure (Fig. 3) (2, 5).

Fig. 3. Earthquakes at the site of the Ingouri dam during May 1978 (5). Triangles - seismic stations.

In October 1978 the reservoir was filled up to 170 m, i.e. to the design level of the 1st stage Ingouri reservoir. As this took place the gap of seismic activity occurred from August to December and from December 1978 again a rise in seismic activity was observed.
In this case migration of foci of earthquakes to South-West of the dam towards the bend of the Tskvarcheli flexure was observed. In the following 9 months single earthquakes $K = 8-9$ were confined to that zone.

During three months a gap in the seismic activity was observed and then in the region of the flexure bend activity rose sharply and at a distance of 18-25 km from the dam site a large swarm of earthquakes occurred, the strongest earthquakes of which - of December 21 and 27, 1979 - had energy class $K = 12$ ($M = 4.2 \pm 4.3$) and manifested on local area with macroseismic intensity 7 degrees according MSK-64 scale (Fig. 4). Depth of foci intensity 6 degrees is 190 km$^2$, at the construction site the macroseismic effect did not exceed intensity 4 degrees. The level of seismic activity $A_{10}$ increased up to 22 during this period but the value of $\chi$ fell down to 0.2-0.32.

The source mechanism of the strongest shocks is a strike-slip displacement in a sublatitudinal direction which is different from common case mechanisms for the given zone.

In 1980-1981 seismic activity in the area of the reservoir lowered down markedly, however during this period in the vicinity of the reservoir single earthquakes of $K=10$ occurred. The level $A_{10}$ during that period was somewhat higher compared with the level before the filling of the reservoir.

Evaluating the seismic situation in the area of the Ingouri Project as a whole for the period of observations it should be noted that the swarm of earthquakes in December, 1979 - January, 1980 with a magnitude of strongest shocks of up to 4.4 is the most notable seismic event in this region since 1972. However, taking into account seismic history of last decades this is not an exceptional phenomenon compared with the Megrelo-Svansikoe earthquake of 1930 ($M = 4.8$), the Gegechkorskoy swarm of earthquakes of 1957-1958 ($M = 5.3$), etc.

Energy and microseismic occurrence of the Ingouri swarm of earthquakes is in a good line with the evaluation of seismic rise which was adopted for the designing of the Ingouri reservoir. Space and time confinement of the reservoir filling bears witness to the connection of this swarm of earthquakes with the reservoir. However taking into consideration rapid attenuation of the swarm and a fall of the slope of the recurrence curve the character of these earthquakes is tectonic and the filling of the reservoir approached the moment of origin of the Ingouri swarm of earthquakes only. As a whole the events were prepared by the preceding course of tectonic development in this region and they occurred in the weakest and most deformable areas of the earth's crust.

The results of precise levelling of the valley section of the

Fig. 4. Earthquakes of the Ingouri swarm for the period of December 21-31, 1979 (5): 1 - seismic stations; 2 - energy classes of earthquakes $K = 1g B_{1,2}$; $K=1.8M+4.0$; 3 - regional faults.
Ingourri river of about 50 km long including the section stretching for 12 km downstream of the dam and the section of about 38 km long upstream of the dam up to the reservoir tail are of particular interest. Before the filling of the reservoir the section downstream of the dam and stretching for 5 km upstream of the dam had a total rise of up to 35 mm in a year and this value fell down towards the dam site. In 1978-1979, i.e. the first year after creation of the reservoir, the process of the rise was rather ordinary, e.g. for some 10-20 mm in the gorge of the Magana river - the left-bank tributary. In 1979-1980, i.e. the year after creation of the reservoir a step-like rise of elevations for some 20-25 mm took place at the upstream section stretching for 5-6 km at 4-5 km from the dam and a fall of levels for some 20-35 mm occurred at nearby upstream sections and at the section located at 12 km downstream of the dam. The boundary of a step-like drop of elevations lies in the head section of the reservoir at 4-5 km from the dam.

Is there any connection between this phenomenon and the creation of the reservoir, and what is the effect of this phenomenon on reliability of the reservoir? These questions are being studied now.

A dense system of reliability control of the Ingourri reservoir apart from its main purpose, i.e. evaluations of conditions and prediction of behaviour of the structure makes it possible to reveal and study a series of new phenomena related to development of deformation and seismotectonic processes in the rock mass of the reservoir zone.

Conclusions

1. For studies of deformation processes of different intensities taking place in the earth's crust at the site of the Ingourri hydroelectric station a detailed system of geophysical, seismic and geological observations was set up. The tasks of this system comprise the following:

   a) to reveal and study the most deformable sections in the rock foundation of the structure where deformations dangerous for the structure are likely to occur;
   b) to determine the effect of the reservoir on seismic regime within the zone of the reservoir;
   c) to study the reasons and dynamics of induced ("dam") earthquakes;
   d) to study the behaviour of the foundation and the dam by earthquakes action.

2. Deformation phenomena were found in the near-surface section of the earth's crust in the headwater section of the Ingourri reservoir and in the area of the reservoir effect (in the radius up to 30 km from the dam). These phenomena were associated with seismicity even at the initial stage of the reservoir filling (the head of some 100-170 m). Deformation processes in the zone of intersection of the Ingiresh fault and the faults of the Tkvarcheli flexure (the headwater portion of the reservoir) gave rise to the swarm of induced earthquakes with M = 1.1-1.6 in a month after the reservoir filling.

The creation of the Ingourri reservoir favoured the activity of deformation processes in the zone of the bend of the Tkvarcheli flexure (at 20 km from the dam site) as a result of which a powerful swarm of earthquakes developed in this zone after 20 months had passed since the filling of the reservoir. The strongest earthquakes of the swarm were of magnitude M = 4.2-4.4 and felt in the epicenter area with intensity up to 7 degrees of MSK-64 scale.

3. Detail comprehensive geophysical, geodetic and seismic studies under way now allow to obtain new data about development of deformation and seismotectonic processes in the earth's crust in the zone of the deep Ingourri reservoir (depth at the dam is 270 m) in the course of completion of the Project.

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THE DETERMINATION OF THE SEISMIC CHARACTERISTICS OF CANALES DAM SITE—
GRANADA (SPAIN)

LA DETERMINATION DES CARACTERISTIQUES SISMIQUES DE L’EMPLACEMENT DU
BARRAGE DE CANALES—GRANADA (ESPAGNE)

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ABSTRACT

The methodology used in the definition of design earthquake of Canales dam
site is exposed, analysing also some more general aspects relating to the parti-
cular problems of seismotectonic studies.

RESUME

On explique la méthode à suivre pour la définition du sisme de calcul
dans l’emplacement du barrage de Canales, en exposant aussi quelques aspects plus
générales de la problématique des études sismothectoniques.

INTRODUCTION. AMBIT AND METHODOLOGY OF THE STUDY

Canales dam, at present under construction, is situated on Genil river, a few kilometers upstream from the
city of Granada. Geologically the zone corresponds to the Betic mountain range, chain of alpine age, and belongs
to one of the areas of greater seismicity in the Iberian Peninsula. This
fact, the dam’s typology and height—rock fill, 156 m. height—so as the
recent development of adequate methodologies on the calculation of seismic
parameters, previously applied and

checked in Spain in other kinds of works such as nuclear power plants, have moti-
vated the Administration’s decision to analyze in detail the calculation seism-
ic parameters instead of applying, as
has been the case to date, the values
duced directly from the specifications
indicated in the Spanish Seismoresistant
Standards, which have a very general
character. The seismic calculation for
Canales dam is the first to be underta-
kenn in Spain with these criteria.

It is known, in accordance to empri
cal data verified in different parts of the world, that the study of seismi-

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city affecting a determined work site, usually is extended to a circular area 300 km. in radius centered on the work, where the different epicenters and their incidence on the considered site are investigated. However, in high seismicity zones it is possible to reduce the acting area, since the remote foci will determine, in any case, smaller effects on the studied site than nearer ones, as much as due to their distance to the work as for their own seismicity. In particular if the pursued calculation method is the deterministic one, since in this case actual occurred phenomena are used and it is not necessary to establish the fitting recurrent curve, like it occurs in the probabilistic method. In this last case the correct definition of the indicated curve will require the analysis of remote epicenters in order to arrive at a suitable representation of the phenomena sample, especially of its lower part which will be mainly defined by the more distant foci.

The authors' experience in the realization of other studies similar to this present, where both calculation methods, deterministic and probabilistic, have been pursued, shows that, in different points of the Spanish geography, the deterministic calculation intensity corresponds to probabilistic recurrence periods varying from less than 500 years to something over 900 yards (Sáenz Rínduejo, C.; Arenillas, M. & López Marinas, J.M., 1978).

All the preceding items have induced that in the Canales dam seismic parameters determination, the zone studied has been restricted to a circular area 100 km. radius centered on the site, adopting the deterministic method. The study has been divided into two main phases. Firstly the geology of the area has been investigated, putting special emphasis in tectonics, and analysing seismology secondly, with the design earthquake deduction. These aspects constitute the basis of the present paper. The final part includes a section indicating some criteria in relation to the subsequent phases of the seismic calculation.

As far as the probabilistic method is concerned, which is not contemplated here, an analysis of same applicable to Spanish dams can be seen in Arenillas, M., Sáenz, C. & Samartín, A. (1982).

**GEOLOGY**

The Betic chains form a mountainous line to the south of the Iberian Peninsula, orientated in WSW-ENE direction, extending from Cadiz gulf through most of Andalucia, Murcia and South of Valencia. Its length is 600 km., with variable width, less than 200 km. Inside the Peninsula, the boundaries to the North are the "Meseta" and the "Iberian Cordillera"; towards the South the Mediterranean sea, from Cadiz to Cullera (Valencia). The Meseta and the Betic mountain range are separated to the West by the Guadalquivir valley.

The Betic chains are part of the Alpine Mountain Belt, which many authors indicate that they continue in North Africa, by the Rif and Tell alignment. Major structural characteristics are known by reflexion seismic prospecting data, which especially show the
existence of a large subduction zone between Gran and Marroco plates, produ-
cing shortening over the marine surfaces and causing the Betic materials to be folded, fractured and piled towards the North by several sliding and gravity nappes,olistostromes and slumping phe-
omena in the Guadalquivir valley.

Betic mountains can be divided in two major groups: External Zones and Internal Zones. The first with folded cover structures and overthrusting, the later with stronger deformation which includes the basement and is accom-
panied by metamorphism. Furthermore these large zones, the "Campo de Gibral-
tar" and the "Intermedial Units" can also be differentiated. Each one of these major zones or units present the possibility of being divided in others of lower rank (fig. 1).

Besides these structural units there are postorogenic materials filling the intermountain basins (Antequera, Granada, Guadix—Baza, Lorca, Murcia) and volcanic rocks, andesites and trau-
quites, from the Neogene-Quaternary of Gata cape.

Concerning to the study area, the tectonic units which can be considered according to Julivet et al. (1971) and Fallot classic division are:

a) Prealpidic Units

They are formed by materials from the Internal Zones or Betic s.s., in which the Nevada-Filabride, Alpujarride and Malaguide complexes are included. They are the oldest terrains of the Betic chain and have suffered at least one pre-alpidic orogenesis; nevertheless the origin of the region principal struc-
ture is the alpidic-age sliding nappes tectonic.

b) Mesozoic and Tertiary terrains deformed by the alpidic orogenesis

These terrains come from the External Zones where it is usual to differentiate many tectonic units related to sliding nappes and overthrusting. Generally these units coincide with the paleogeographic ones.

The general disjointing over a Paleozaic basement, which does not emerge, is due to Keuper plastic materials, nowadays covering important extensions of the northward Mesozoic age units. In these typical sliding nappes structures, with overthrusting scales over the Meseta edge, tectonic windows and blocks in lands are frequent.

c) Terrains with Cretacic and Eocene deformations

They include several units from the Sub-Betic (External Zones), folded and slid during Cretaceous or Eocene, coinciding the stratigraphic and the tectonical units. From South to North the different zones are: Internal Sub-Betic, Medium Sub-Betic and External Sub-Betic. The three of them can be divided in smaller units.

d) Terrains mainly deformed during Upper Oligocene and in a minor way during the Miocene

This period deformed terrains be-

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and, in some places, to the Malaguide Complex, being the later of little importance.

e) Gravity translated terrains during Upper Oligocene to Lower Miocene

They are located on a depressed zone between the ultra-internal Sub-Betic and the Malaguide Complex, containing principally Cretaceous or Nummulitic terrains, although in some spots, outside the studied area, older terms Jurassic and Triassic can appear.

f) Terrains mainly deformed during Lower and Upper Miocene

Inside the External Zones, the Miocene deformed materials are Pre-Betic series overthrust by the Sub-Betic. The joint between Pre-Betic and Sub-Betic is marked by a huge strike-slip fault, NW-SE direction, sensibly perpendicular to the folds strike. The numerous thrust scales, piling in sliding thrust surfaces, are nearly horizontal, nearby the Meseta basement. These surfaces bend progressively towards the Internal Zones, where fractures and faults increase.

g) Guadalquivir Valley terrains, gravity translated during Lower Miocene

This Lower Miocene material comes from the Internal Zones. The original substratum is formed by white siliceous Bourdigalian marls, extending from the Guadalquivir Valley to the Prebetic, where they occupy huge extensions.

h) Terrains deformed during terminal Miocene

They correspond to Neogene terrains more recent than the sliding tectonic. They sedimented after a strong erosion period, and fossilized reliefs created in previous orogenic stages. They cannot be declared as postorogenic, because they have been slightly affected by late tectonic episodes. Recent folding stage affecting the region is from Tortonian beginnings. This stage is followed by strong fissuring; and normal faults formation over the edges of the basins.

The region present seismic instability shows a fractured blocks tectonic, which affects the whole Western Mediterranean. This tectonic has its own trends, independent from the Alpidic one.

The Canales dam lies over molasses and "maciños" (marlous sandstones) from the Helvetiense, which are in discordance over older silty clay terms, being joined with the Alpujarride Trias by a fault. This fault, or better group of faults, extends through all the Alpujarride-Neogene contact, by the Western edge of the Granada basin, showing activity even at present.

i) Post-tectonic Units

They are unfolded Neogene and recent materials. They correspond to filling deposits of the internal basins, as well as to the original sediments from the Guadalquivir Valley. Although not interesting from the tectonic point of view, they do have interest because they fossilize faults and fractu-
res, covering units which were of great importance during folding and sliding periods.

SEISMOLOGY

In Spain, as in many other countries, an abundant historical information is available, that has allowed the catalogation of numerous earthquakes. The Spanish seismic record includes, more or less in detail, a period of almost 2,000 years whose information have been collected in several reports. The most complete ones are these by Galbis (1932, 1940) and Munuera (1963) used as a basis, with other more dispersed data, to prepare a general record by the Instituto Geográfico Nacional (I.G.N.). The earthquakes collected there are fundamentally characterized by the intensity, since logically the instrumental magnitude is known only with reference to the modern events. Independently of the precisions which will be mentioned later, the general I.G.N. seismic catalogue permits the establishment of the adequate seismic map of the studied zone. This map together with the geological, especially tectonical characterization, as previously said, has allowed in the present case, the division of the area in a series of seismotectonic units (fig.2).

It is evident that the mentioned seismotectonic division could not be an unique one. In certain cases the tectonic knowledge on determined zones will be scarce, in others the seismological characterization would be imprecise. All this guides to subjective appraisals that, undoubtedly, will influence on the final result. The method’s detractors show examples where several specialists reach clearly different conclusions related to a same zone. Evidently this could be the case, as also could be the disparity in the results of the flood calculation of a determinative, as in other many more cases in civil engineering. Nevertheless, when it is possible to part from the same basic information and the facts are treated by people with the adequate knowledge on the matter, the experience shows that the final results—in our case the value of designing earthquake—are practically the same one from the other, and all of them are within the allowable limits in civil engineering, where obviously it is not intended to arrive at an exact value in absolute sense, but to a figure which results coherent with the problem to be solved. Another question would be the different kind of the basic information available for varying cases or the possibility that unqualified persons compile the data analysis; the later case is unfortunately more frequent than desirable.

In the case now considered, the available seismotectonic information is quite complete, permitting a division of enough detail, only posing some doubts in a few very precise points. For example it has been necessary to group all the marine areas in one sole unit, due to scarce geological and seismic data to permit a greater degree of division. Also, with a wide criteria, the decision has been taken to group in another unit all the external zones of Betic chains, since their low seismicity, in relative terms, and their dis-
tance to the site, determine, in any case, an incidence of minor order on the work and, consequently, one would not reach a better deterministic results with a greater subdivision.

The assignment of corresponding characteristic seismicity to each of these units is a more complex problem. Firstly the question arises of the much discussed theme in connection with the parameter to be used, and secondly the validity of available seismic data. As far as this last aspect is concerned, it is necessary to consider that not all of the 788 earthquakes included in the I.G.N. list as occurred in the studied area till 1975, last date appearing in the official dossier, have been investigated with the same accuracy by the authors of the different catalogues used in the preparation of the mentioned list. This leads to show the existence of errors in certain cases, aspect already verified in previous studies (Sáenz Hiduejo, C. et al., op. cit.). This fact requires, in general, a detailed analysis of the greater part of the historical earthquakes included in the official catalogue. However in Canales study, having followed the deterministic calculation method, it has been sufficient to investigate only the principal earthquakes of each seismotectonic established units. This has meant the detailed study of 24 earthquakes, proving the validity of the data referred to 14 of them; on the contrary it has been necessary to vary the intensity and the localization of one earthquake, the localization of another three, the intensity of one more and, finally, it has been noticed that five of them resulted inexistent or very doubtful and, consequently, inapplicable to the study.

In the Spanish case the critical analysis of the different data compiled in the catalogues, excluding the very old earthquakes, is an arduous but accessible task, considering the abundant documentation normally existing in general, local, parochial, etc., archives, to which it is necessary to refer in order to correct or complete the information appearing in the mentioned general lists.

This last question is also related to the other subject previously mentioned, namely to the decision regarding the parameter to be used in the characterization of the seismicity of each seismotectonic unit. The problem is based on the choice between intensity and magnitude. In the Spanish case, our opinion (explained in previous occasions: see, for example, Sáenz Hiduejo et al., op. cit.) is that it is necessary to use, undoubtedly, the intensity, at least in this first phase of the study, since to resort to the magnitude supposes leaving aside a large part of the available sample, or to make excessive hypothesis about the own characteristics of each catalogued historical earthquake.

According to these previous criteria, in Canales dam study a determined characteristic intensity has been assigned to each considered seismotectonic unit. These values vary from IV to X (M.S.K.), with the predominance of the greater figures (I > V II) which is justified by the high intensity correspon-
ding to the Betic mountain range as it has been already said.

In accordance with the deterministic calculation method, the nearest points to the site of every unit have been taken as characteristic epicenters of each of them. This leads to situation in the site itself the characteristic epicenter of the unit in which the dam is located. The incidence of other units has been analyzed studying the seismic transmission from respective characteristic epicenters. This question is not yet sufficiently solved, at least in Spain, due to lack of seismic damping curves of general character applicable to the different earthquakes in connection with their localisations and focal mechanisms. To solve this problem it is necessary to resort to a few damping curves, already deduced or deducible from the iso-seismal distribution curves of some well-known earthquakes, and then apply these relationships to the different characteristic earthquakes of every seismotectonic unit. Concerning this point, it should be noticed, on the one hand, that the better defined earthquakes that allow good deductions of seismic damping, are usually the most destructive and the effects of which have extended to greater areas; on the other hand that the slower damped actions with origin in a particular epicenter and for the same epicentral intensity, correspond to greater magnitude phenomena. That is to say, generalising the attenuations of these greater earthquakes to the others, one would be working on the safe side, which with a conservative criteria, allows and justifies in a certain manner, an extrapolation like that indicated. With these criteria, in Canales dam seismic calculation the damping curves have been deduced from six well studied earthquakes, applying these transition types to the characteristic earthquakes of the different seismotectonical units (fig. 3). In accordance with this and with deterministic method general principles, the seismic intensities that foresseably will reach Canales dam site during the work useful life, on account of the characteristic earthquakes of the different seismotectonic units, will be those indicated on the table num. 1.

Proceeding results indicate that the determinant seismicities on studied site are those corresponding to the work unit itself and to Sierra Almijara unit. In both cases a calculation intensity of VIII (M.S.K.) degree is defined. The obtained value is evidently high but coherent with the site seismotectonical ambit. The following factors should not be forgotten, amongst others: a) the high seismicity of the Betic chains, especially of their internal zone, which is where the work is located; b) the existence of the great Andalucia earthquake of 25.12.1884, produced in Sierra Almijara unit, in connection with which a large amount of historical documentation is available; c) the fact that different VIII (M.S.K.) degree earthquakes in the work unit itself have occured previously, whose defining characteristics are well documented.

SEISMICAL ACTIONS ON THE SITE

The next step in the calculation
Fig. 3: Adopted damping curves
TABLE NUM. 1

<table>
<thead>
<tr>
<th>Unit</th>
<th>$I_n$</th>
<th>Distance (km.)</th>
<th>Intensity at site</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Granada</td>
<td>VIII</td>
<td>--</td>
<td>VIII</td>
</tr>
<tr>
<td>2. Alhama</td>
<td>VIII</td>
<td>16</td>
<td>VII</td>
</tr>
<tr>
<td>3. Occidental Alpujarride</td>
<td>VI</td>
<td>0.5</td>
<td>VI</td>
</tr>
<tr>
<td>4. Albuñuelas</td>
<td>V</td>
<td>21</td>
<td>III</td>
</tr>
<tr>
<td>5. Sierra Nevada-Filabres</td>
<td>VI</td>
<td>4.5</td>
<td>IV</td>
</tr>
<tr>
<td>6. Guadix</td>
<td>VI</td>
<td>21</td>
<td>I</td>
</tr>
<tr>
<td>7. Santa Barbara</td>
<td>IV</td>
<td>43</td>
<td>II</td>
</tr>
<tr>
<td>8. Baza</td>
<td>VI</td>
<td>67</td>
<td>III</td>
</tr>
<tr>
<td>9. Lucar</td>
<td>VIII</td>
<td>85</td>
<td>II</td>
</tr>
<tr>
<td>10. Malaga</td>
<td>X</td>
<td>68</td>
<td>VII</td>
</tr>
<tr>
<td>11. Sierra Almijara</td>
<td>X</td>
<td>39</td>
<td>VIII</td>
</tr>
<tr>
<td>12. Lujar</td>
<td>VII</td>
<td>22</td>
<td>VI</td>
</tr>
<tr>
<td>13. Albuñol</td>
<td>VIII</td>
<td>30</td>
<td>VI</td>
</tr>
<tr>
<td>14. Andarax-Canjayar</td>
<td>V</td>
<td>39</td>
<td>III</td>
</tr>
<tr>
<td>15. Gador-Dalias</td>
<td>X</td>
<td>41</td>
<td>VII</td>
</tr>
<tr>
<td>16. External Betics</td>
<td>VIII</td>
<td>18</td>
<td>V</td>
</tr>
<tr>
<td>17. Marines</td>
<td>VII</td>
<td>47</td>
<td>IV</td>
</tr>
</tbody>
</table>

Another way would be to work on focal magnitudes with the opportune correlations. Nevertheless, as mentioned previously, the imprecisions due to this method would not allow the final results to be improved objectively.

For these reasons in the Canales dam case and in this first calculation phase, which only tries to determine the site seismical characteristics, independently from the dam adopted structural solution, it was decided to use, as a first approximation, some of the existing intensity/acceleration relationships. From these basis (fig. 4) it would result that the acceleration on site could vary between 0.05g and a little over 0.30g, with a maximum to minimum relationship of 6 approximately, which is evidently inadmissible. With a conservative criterion it would be ne
Fig. 4: Relationships between intensities and acceleration according to different authors
cessary to resort to one of the higher values of the mentioned relationships. In Spain, with this point of view, and in especially problematic works as the nuclear power plants, it has been usual to employ Neumann's curve. This relationship, in the vicinity of $I = \text{VIII}$ considered in the Canales case, shows acceleration values slightly higher than those appearing in the more recent relationship by Murphy and O'Brien, which has been obtained from a very wide earthquake sample, aspect which should be considered. On the other hand, the Japanese standard on this matter, which uses numerous contrasted data, establishes for rock fill dams, which is the type adopted in Canales, seismic parameters values varying from 0.12 to 0.20, in equivalent or higher seismicity areas than that deduced in Canales.

All this, although being conscious of existent imprecisions, has induced the use in Canales dam seismic calculation of an acceleration value on the site of 0.20g, as a "quantifying" element of the previously deduced $I = \text{VIII} (\text{M.S.K.})$ intensity. This figure is evidently a first approximation, remaining for the following calculation phases the establishment of adequate correcting parameters, in accordance to the structural solution adopted.

Therefore it should not be forgotten that, in general, a figure like the previous one, has little meaning out of the outlined problem's global context. Aspect which is often ignored, with the obsession -so in vogue today- to find a figure, more "exact" better it is, justifying extensive and prolix calculations. At present no method seems to exist to allow the characterization of seismicity affecting a determined work by a unique figure applicable directly to structural calculation. The factors related with the problem and which will remain as real or potential damages are numerous: accelerations, velocities, displacements, frequencies, seisms duration, etc.; up to now the seismic intensity is the only parameter that allows the effect of all these factors to be grouped together. Now then, this parameter could not be introduced in structural calculation other than on a series of later elaborations taking into account all the previously mentioned factors, relating them directly to the structural solution studied. The definition of the design intensity is but a first phase in the problem's resolution. But this phase is a basic one and should not be left aside, or simplified by a sole correlation with any of the mentioned parameters, as could be the acceleration.

Consequently in Canales dam seismic characteristic study it is only possible to point out that the maximum deterministic intensity to be considered in the work is of VIII (M.S.K.) degree and corresponds to a near focus. The parameters which should be considered in the structural calculations should be derived from this basis, considering the seismotectonical ambit of the site and the work's own characteristics. This question is excluded from this paper. To take into account a distant focus is also a problem of very particular characteristics. Firstly it
would be necessary to define what is understood as near or distant and which are the geological conditions allowing this kind of differentiation. We do not enter into this subject either.

REFERENCES


SEISMIC STUDIES FOR THE DYNAMIC TESTING OF ROCK FORMATIONS AT BEAS AND THEIR DAM SITES, PUNJAB, INDIA

DES ETUDES SISMQUESTES POUR L'ESSAI DYNAMIQUE DES FORMATIONS DE ROCAILLE A BEAS ET AUX EMPLACEMENTS DE BARRAGE THEIN, PUNJAB, INDE

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ABSTRACT

Dynamic measurements of elastic parameters of different rock units of younger Tertiary Siwalik Formations at Beas and Thein Dam sites were carried out by seismic method. Short refraction seismic profiles run over different rock units enabled the measurements of compressional and shear wave velocities which in turn permitted the determination of Poissons ratio and other elastic parameters of bed-rock units. These measurements were conducted over clay-shale, sandrock, and the imper- visous core material at the Beas Dam site and sandstone and claystone at the Thein Dam site. The paper discusses in detail the testing procedure adopted, instruments used, the comparative values of Poissons ratio and modulus of elasticity obtained and the geological evaluation of test results. It is hoped that the data on dynamic measurements, which have advantage over the static measurements in that a large rock mass is sampled and the parameters obtained are more representative, made available by these studies will be useful in solving practical engineering problems.

ABSTRACT

Des mesurages dynamiques des paramètres élastiques des différentes unités rocheuses des formations tertiaires Siwalik plus jeunes aux emplacements des barrages à Beas et à Thein ont été faits par la méthode seismique. Des profils seismiques de courte refraction courant sur les différentes unités rocheuses ont permis les mesurages des vitesses de compression et de vague de cisaillement, qui, à leur tour, ont permis la détermination du rapport Poissons et d'autres paramètres élastiques des unités des roches de fond. Ces mesurees ont été faits sur des schistes argileux, sur des grès et sur le matériau du noyau imperméable à l'emplacement du Barrage Beas et sur le grès et sur la roche argileuse à l'emplacement du Barrage Thein. Cette étude parle en détail de la procedure d'essai adoptée, des instruments utilisés, des valeurs comparatives du rapport Poissons et du module d'élasticité obtenu ainsi que de l'évaluation géologique des résultats de l'essai. Nous souhaitons que les données sur les mesurages dynamiques, qui ont un avantagesur les mesurages statiques en ce qu'une large masse rocheuse est échantillonnée et que les paramètres ainsi obtenus sont plus représentatifs, présentés par ces études n'avéreront utiles.
INTRODUCTION

In foundation engineering problems the information on the elastic parameters of different rock formations is found very useful in assessing the nature of rocks, their load bearing capacity and other mechanical properties. A knowledge of the elastic constants is also useful in solving practical engineering problems associated with drilling, blasting of rocks, tunnel excavation and lining. The dynamic moduli of elasticity are the instantaneous deformation moduli under the natural state of stress and would best define the ground response to vibratory loading.

The dynamic testing involves application of routine seismic refraction technique for measuring the Compressional (Primary) and shear wave velocities over different rock units. The determinations, in turn, provide the information on different elastic moduli as their exists relationship between them.

WAVE VELOCITIES VS ELASTIC MODULI

The values of different elastic parameters are calculated from the Primary and shear wave velocities (\( V_p \) and \( V_s \) respectively) using the following empirical relations:

\[
\sigma = \frac{1-\frac{V_p^2}{V_s^2}}{1-\left( \frac{V_p}{V_s} \right)^2} \quad \ldots \ldots \ldots (1)
\]

\[
E = \frac{V_p^2 \rho (1+\sigma) (1-2\sigma)}{(1-\sigma)} \quad \ldots \ldots (2)
\]

\[
K = \frac{E}{3(1-2\sigma)} \quad \ldots \ldots (3)
\]

Where

\( \sigma \) = Poisson's ratio

\( \rho \) = Density of the medium in gm/cm\(^2\)

\( E \) = Young's modulus in dyne/cm\(^2\)

\( K \) = Bulk modulus in dyne/cm\(^2\)

\((10^{10} \text{ dyne/cm}^2 = 1.02 \times 10^4 \text{ kg/cm}^2)\)

SEISMIC STUDIES

The studies for the dynamic measurements of elastic parameters at the two dam sites were conducted by the Geological Survey of India. A facsimile Hammer Seismograph (Model FS - 3, Huntex, Canada) was used to record the seismic data. A 14 lb hammer coupled with steel plate was used as the source for the generation of seismic energy while a three component geophone was deployed as receiver for picking up separately compressional (P) and shear(s) wave refractions. In case of Thein dam site measurements, however, a vertical component geophone only was used as receiver and the shear wave arrivals were identified by the distinct pattern or low order of velocity of later cycles which follow the first few cycles of 'P' wave. (Hobson' 69). A plot of the onset of 'P' and 'S' wave arrivals against the distances of hammer points from geophone gives their velocities. (Figs 1 & 2). In some cases, while working with only vertical component geophone, it was found that the onset of 'S' waves is masked by the later cycles of 'P' wave and, as such, 'S' wave velocity could not be estimated.
The refraction seismic profiles were laid along the alignment of excavations in exploratory tunnels and drift and along the road cuttings. The geophone was kept fixed at a point by planting it inside the rock making some small hole therein and the hammer points were located at distances 5, 10, 15, 20, ..., 45 and 50 m depending upon the availability of space. The profiles were repeated when using a three-component geophone, selecting the vertical and horizontal motion sensitive elements separately. Caution was observed to keep the profiles aligned in the strike direction of the formations wherever possible.

LOCATION OF MEASUREMENTS

The field studies for the determination of elastic constants of different rock formations in the Beas dam area were made only at (1) Intake Bench, (2) Key Trench area and (3) along the dam axis. The measurements inside the tunnels were not possible as all the tunnels were lined up.

At the Thein dam site, the dynamic measurements were made at (1) Right bank exploratory drift No. 1 (DR-1) Old dam axis; (2) Right bank exploratory drift No. 2 (DR-2) Old dam axis; (3) Left bank drift No. 1 (DL-1) Alternative dam axis; (4) Left bank drift No. 3 (DL-3) Alternative dam axis and (5) Road cuttings near the alternative dam axis.

DISCUSSION OF RESULTS

A. Beas Dam Site:

The dynamic tests at the Beas dam site have been conducted over clay-shale, sand rock and the
dam core material. A total of 10 measurements were performed at the Beas dam site and the results are presented in the Table 1.

The clay shales are characterised by primary Wave Velocity (Vp) of 1875 - 2000 m/sec, the shear wave velocity (Vs) being 900 - 1150 m/sec. However, the test No. 8 has revealed the corresponding velocities as 660 m/sec and 360 m/sec for the clay shale encountered at spill way channel cut-slope which was quite loose and severely shattered due to blasting. The sand rock exhibits wide variations from 840 - 1750 m/sec. and 480 - 900 m/sec in Vp and Vs respectively. Very low compressional wave velocities (520 - 570 m/sec) have been indicated for the dam core material. There was some ambiguity in recording the shear wave velocity for the dam core material as the velocities obtained even from the 3rd to 5th cycles range from 300-350 m/sec. This could be the order of velocity for the sound waves and the shear waves for this material characterised by velocity lower than sound waves could have not been recorded.

The value of Poisson's ratio obtained for the clay-shales varies from 0.26 to 0.35 and that for the sand rock ranges from 0.25 to 0.31 (Singh, 1972-73).

Assuming 300-330 m/sec to be the order of Vs for the dam core material, three values of the Poisson's ratio are found as 0.20, 0.25 and 0.29.

The modulus of elasticity (E) has been determined as 0.522 to 0.785 x 10^5 kg/cm^2 for clayshale.

A very low value of 0.380 x 10^5 kg/cm^2 was also obtained in test No. 8 (Table 1) for the loose and shattered clay shale. The value of E for the sand rock is found
<table>
<thead>
<tr>
<th>Profile No</th>
<th>Location</th>
<th>Formation</th>
<th>$V_p$ m/sec</th>
<th>$V_s$ m/sec</th>
<th>$\rho$ gm/cm$^3$</th>
<th>$\sigma$ $10^5$kg/cm$^2$</th>
<th>$K$ $10^5$kg/cm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>Intake Bench</td>
<td>Clay shale</td>
<td>1875</td>
<td>900</td>
<td>2.35</td>
<td>0.35</td>
<td>0.522</td>
</tr>
<tr>
<td>B-2</td>
<td>Intake Bench</td>
<td>Sand rock</td>
<td>1750</td>
<td>900</td>
<td>2.35</td>
<td>0.31</td>
<td>0.532</td>
</tr>
<tr>
<td>B-3</td>
<td>Intake Bench, Top of clay-Shale Band No 1.</td>
<td>Clay Shale</td>
<td>2000</td>
<td>1150</td>
<td>2.35</td>
<td>0.26</td>
<td>0.785</td>
</tr>
<tr>
<td>B-4</td>
<td>Dam Axis, Dam Stn. 69 Core Chain 30, material Elevation-133\frac{1}{2}</td>
<td></td>
<td>520</td>
<td>320(?)</td>
<td>2.16</td>
<td>0.20</td>
<td>0.054</td>
</tr>
<tr>
<td>B-5</td>
<td>Key Trench Area (N-S)</td>
<td>Sand rock</td>
<td>840</td>
<td>480</td>
<td>2.35</td>
<td>0.26</td>
<td>0.138</td>
</tr>
<tr>
<td>B-6</td>
<td>Key Trench Area (W-E) with bands of clay shale lying conformably</td>
<td>Sand rock</td>
<td>1125</td>
<td>650</td>
<td>2.35</td>
<td>0.25</td>
<td>0.252</td>
</tr>
<tr>
<td>B-7</td>
<td>Spillway Channel cut slope</td>
<td>Sand rock</td>
<td>1060</td>
<td>650</td>
<td>2.35</td>
<td>0.23</td>
<td>0.232</td>
</tr>
<tr>
<td>B-8</td>
<td>Spillway Channel cut slope</td>
<td>Clay Shale (loose and shattered)</td>
<td>660</td>
<td>360</td>
<td>2.35</td>
<td>0.29</td>
<td>0.030</td>
</tr>
</tbody>
</table>

VIII.285
is found varying from 0.138 to 0.532 x $10^5$ kg/cm$^2$. The values of $E$ & $k$ as determined for the dam core material vary from 0.051 to 0.060 x $10^5$ kg/cm$^2$ and 0.030 to 0.040 x $10^5$ kg/cm$^2$ respectively. As these parameters for the dam core material have been calculated presuming the velocity of shear waves which is identical to those of sound waves, the figures arrived at are to be used with caution.

B. Thein Dam Site:

The dynamic measurements at Thein dam site were conducted over the sandstones and claystones excavated as drifts at the old dam axis and the alternative dam axis. A total of 8 tests were made and their results are furnished in Table - II and discussed below location-wise:

Old Dam Axis - I

Near the old dam axis, three measurements were made in two of the exploratory drifts (DR-1 and DR-5) on the right bank which were excavated over a total length of about 35.0 and 50.0m respectively. The tests have revealed that the sandstones excavated near the old dam axis are characterised by $V_p = 3100 - 3200$ m/sec, $V_s = 1650 - 1800$ m/sec. Poisson's ratio = 0.25 - 0.33, $E = 2.024 - 2.206 	imes 10^5$ kg/cm$^2$ and $k = 1.471 - 1.985 	imes 10^5$ kg/cm$^2$.

Alternative Dam Axis (1)

A total number of five dynamic tests were made near the alternative dam axis. Profiles $T_4$ and $T_5$ were laid over the claystone in the left bank drift No DL-1 and the profile T-7 was observed along the road cutting through claystones.

The Primary and shear wave velocities for the claystones are estimated to range from 2450-2500 m/sec and 1150 - 1200 m/sec respectively. They are found characterised by Poisson's ratio 0.35, the modulus of elasticity in the range of 0.971 to 1.032 x $10^5$ kg/cm$^2$ and bulk modulus varying from 1.080 to 1.136 x $10^5$ kg/cm$^2$.

The profile T-6 laid along the strike in the left bank drift No. DL-1 has yielded that the sandstones near the alternative Dam Axis - 1 are characterised by $V_p = 3000$ m/sec, $V_s = 1650$ m/sec. Poisson's ratio = 0.28, $E = 1.939 	imes 10^5$ kg/cm$^2$ and $k = 1.44 	imes 10^5$ kg/cm$^2$. These parameters when compared with those of sandstones near the old Dam Axis indicate
that comparatively higher values of the modulus of elasticity and bulk modulus have been observed over the latter while the Poisson's ratio for the test T-1 and T-6 remain of the same order (Table II).

The measurements in profile T-7 made along the road cutting near the alternative dam axis over claystone have given almost identical results as that of profile T-4 (Table II).

GEOMORPHOLOGICAL EVALUATION OF SEISMIC TEST DATA

A. Beas Dam Site:

Geology of the dam site

The Beas dam site is located on Upper Siwalik Sedimentary formations comprising sandrock and clayshale/siltstone. The sandrock member exhibits low degree of compaction and consolidation and crumbles down in water. The clayshale member, however, is generally massive and well compacted. These rocks dip at 40°-20° and have been involved in tectonic movements resulting in folding. As a result of their participation in the tectonic movements thin plastic gougy seams have developed in various clayshale bands. Lithological variations i.e. lateral thickening and thinning of the bands is prominent here.

The dynamic measurements at the Beas dam project were made at intake bench, key trench area and along the dam axis over the sandrock, clayshale members and the dam core material.

The Poisson's ratio in the case of clay shales varies from 0.26 to 0.36 depending on the degree of saturation. It is observed to be low for the top clayshale layer which is comparatively dry. The modulus of elasticity (E) for the clayshale has been determined as 0.522 to $0.75 \times 10^5$ kg/cm² whereas the loose and shattered clay shale has given low values of $E(0.090 \times 10^5$ kg/cm²). This could be attributed to the interstitial bonding material - the 'Cement'. It is partially cemented at some places and loose at others.

The values of Poisson's ratio obtained for the dam core material as 0.20, 0.25 and 0.29 appear to be on the lower side which could be attributed to it's being saturated and undercompacted aggregate of clayshale and sandrock.

B. Thein Dam Site:

Geology of the dam site

The Thein dam site is located on the Lower Siwalik formations which comprise alternating beds of sandstones, siltstones and claystones having a general dip of about 60° in 3.50°W in downstream direction. In general, the sandstone bands are fine to coarse grained and micaceous. They are massive in nature but are fairly soft and give an earthy sound on hammering. There are, however, thin lenses or bands of hard sandstones within the soft sandstone. They also show wide spaced joints and fractures.

The claystone is generally purple and hard when dry. Fracturing and presence of bedding shear seams are quite common.

A geological account of various tests carried out at the site is given below:

Tests in right bank drifts No DR - 1 and DR - 3.

Dynamic tests $T_1$, $T_2$ and $T_3$ were carried out in these drifts excavated for lengths of 35m and 30m respectively through thick sandstone band dipping at 65° towards 8.25°W direction. The alignments of the drifts are about 5° oblique to the strike of the sandstone band. The sandstone near the portal is slightly weathered but inside the drift, it is fresh and hard in both the drifts. The drift No. DR-1, wherein the Test $T_1$ was conducted, starts
<table>
<thead>
<tr>
<th>Profile No</th>
<th>Location</th>
<th>Formation</th>
<th>( V_p ) m/sec</th>
<th>( V_s ) m/sec</th>
<th>( \rho ) gm/cm(^3)</th>
<th>( \sigma ) kg/cm(^2)</th>
<th>( k \times 10^5 ) kg/cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-1</td>
<td>Right bank</td>
<td>Sandstone</td>
<td>3100</td>
<td>1700</td>
<td>2.7</td>
<td>0.28</td>
<td>2.090</td>
</tr>
<tr>
<td></td>
<td>right bank</td>
<td>Old dam axis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-2</td>
<td>Right bank</td>
<td>Sandstone</td>
<td>3100</td>
<td>1800</td>
<td>2.7</td>
<td>0.25</td>
<td>2.206</td>
</tr>
<tr>
<td></td>
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<td>Old dam axis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-3</td>
<td>Right bank</td>
<td>Sandstone</td>
<td>3300</td>
<td>1650</td>
<td>2.7</td>
<td>0.33</td>
<td>2.024</td>
</tr>
<tr>
<td></td>
<td>right bank</td>
<td>Old dam axis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-4</td>
<td>Left bank</td>
<td>Claystone</td>
<td>2500</td>
<td>1200</td>
<td>2.6</td>
<td>0.35</td>
<td>1.032</td>
</tr>
<tr>
<td></td>
<td>drift No</td>
<td>alternative</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DL-3,</td>
<td>dam axis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-5</td>
<td>Left bank</td>
<td>Claystone</td>
<td>1600</td>
<td>800</td>
<td>2.6</td>
<td>0.35</td>
<td>0.465</td>
</tr>
<tr>
<td></td>
<td>drift No</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DL-3,</td>
<td>dam axis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-6</td>
<td>Left bank</td>
<td>Sandstone</td>
<td>3000</td>
<td>1650</td>
<td>2.7</td>
<td>0.28</td>
<td>1.959</td>
</tr>
<tr>
<td></td>
<td>drift No</td>
<td>alternative</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DL-1,</td>
<td>dam axis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T-7</td>
<td>Road cutting</td>
<td>Claystone</td>
<td>2450</td>
<td>1150</td>
<td>2.6</td>
<td>0.35</td>
<td>0.971</td>
</tr>
<tr>
<td></td>
<td>near alternative</td>
<td>dam axis</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Densities of the rocks have been assumed.
almost at the contact of the sandstone band and the underlying claystone band. By virtue of its downstream dip, the claystone is expected to be present at 1.0 to 1.5m depth below the floor of the drift at the portal and 6 to 7.0m below the floor at the heading.

In the profile T-1 and T-2, the geophones were planted at the heading of the drift DR-1 and DR-3 respectively and in case of profile T-3, the geophone was placed at the portal of drift DR-3. The modulus of elasticity (E) in the three has been worked out to be $2.09 \times 10^5 \text{ kg/cm}^2$, $2.206 \times 10^5 \text{ kg/cm}^2$ and $2.024 \times 10^5 \text{ kg/cm}^2$ respectively. Assuming the variations in these values to be due to the variation in the rock condition alone, the following explanations can be offered.

1. The presence of underlying claystone in the profile T-1 may have given rise to lower value of E.

2. In the case of profile T-3, presence of weathered sandstone near the portal where the geophone was planted, may have given rise to lower value of E.

3. The value of $2.2 \times 10^5 \text{ kg/cm}^2$ obtained in the test T-2 should be taken as the more representative figure for the modulus of elasticity for fresh and sound sandstone.

Test in left bank drift No. DL - 3

The tests T-4 and T-5 were carried out in the drift DL-3 located on the left abutment along the alternative rock-fill dam axis excavated for a length of 46m in S 55°E direction. Purple coloured claystone with minor bands of siltstone are met within the drift. The rock is highly jointed to sheared.

The profile T-4 was laid along the main drift having the geophone at the heading. The modulus of elasticity and Poisson's ratio as computed from this profile are $1.032 \times 10^5 \text{ kg/cm}^2$ and 0.35 respectively and correspond to that of jointed claystone.

The profile T-5, laid in the cross-cut of drift DL - 3, has yielded the value of modulus of elasticity and Poisson's ratio as $0.465 \times 10^5 \text{ kg/cm}^2$ and 0.33 respectively. The lower value of modulus of elasticity in the profile can be expected as it was run across the strike while profile T-4 was approximately in strike direction. Also in the test T-5, the total length of profile was only 9m as a result of which, the depth of penetration of seismic waves was limited to shallow depth (2-3m only) and parameters obtained are attributable to loose rock zone affected by blasting and destressing.

Test in left bank drift No. DL - 1

The drift DL - 1 is made on the left abutment along the alternative 1 B dam alignment. It is excavated in S 50°E direction for a length of 46m and its alignment is about 10° oblique to the general strike of the formation. Moderately jointed sandstone is met with in the drift which has moist walls and roofs due to water seepage.

The profile T-6 laid along this main drift has given values of modulus of elasticity and Poisson's ratio as $1.939 \times 10^5 \text{ kg/cm}^2$ and 0.28 respectively. These values correspond to highly jointed and saturated sandstone and are lower than those recorded for dry sandstone in test T-2.

Test on the road cut

On the left abutment (at about DL 487.6m) the profile No. T-7 was laid on the main road leading to Dhar (32°24'30", 75° 48'00''). A thick dry claystone band was exposed dipping at 55° in S 25°W direction. The profile was laid in the E - W direction which was about 55° oblique to the strike direction of the bed.
The values of modulus of elasticity and Poisson's ratio as recorded in this test were $0.971 \times 10^5$ kg/cm$^2$ and 0.35 respectively. This value of E is found lying in between the values recorded in the tests T-4 and T-5 corresponding to the values obtained along strike and across strike respectively.

CONCLUSIONS

The results of the dynamic tests conducted at the various sites for different rock formations have been furnished. These when compared with corresponding static values are found to be, in general, on higher side. This may be attributed to the fact that the size of sampled mass is significantly different in two techniques. As in the dynamic tests, the sampling volume is large, the average properties of the formations are obtained.

The tests have shown wide variation in the values of modulus of elasticity and Poisson's ratio depending on the degree of saturation, consolidation and cementation of the Siwalik rock units.

The dynamic tests, which are quick and simple, should find wider applications in the field of rock mechanics.

ACKNOWLEDGEMENTS

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REFERENCES


EXPERIMENTAL STUDY ON SOUNGING UNDERGROUND GROTTOES WITH CROSSHOLE SEISMIC METHOD

ETUDE EXPERIMENTALE DU SONDAGE DE GROTTE SOUTERRAINES PAR LA METHODE DE TROUS CROISES SEISMIQUES

HE SONGLING

Wuhan Design and Research Institute of Coal Mine

China

ABSTRACT

The method of crosshole seismic (underground sounds) testing has been very effective for sounding underground grottoes in foundation soil. This paper presents a new method—"relative changes of amplitude attenuation", which may increase the borehole spacing so as to meet the requirement of borehole spacing in engineering geological prospecting. Our rough tests show that, using this new method, we increase borehole spacing by a factor of 2-3 in comparison with using the method of pseudo-velocity. To survey a grotto about 2 m in diameter, for instance, the borehole spacing is limited to about 30 meters, thus economizing a great amount of drilling work. According to the method, we apply a broad frequency emission and a selective frequency receiving in order to find out the regularity of relative changes of amplitude attenuation of waves of a simplex frequency in rock and soil mass. The instruments and other equipments used are simple and are easily operated. No special conditions are needed in making a test. It is convenient to popularize the method in-situ. Our method may be applied to engineering geological prospecting to survey underground grottoes which can't be ascertained by drilling alone.

ABSTRACT

La méthode de trous croisés séismiques (sons souterrains) est très efficace pour le sondage de grottes souterraines dans la fondation du sol. Cet article présente une nouvelle méthode—"les changements relatifs de l'atténuation de l'amplitude des ondes", qui permet l'augmentation de l'espacement entre les trous percés et ainsi satisfaire les exigences de l'espace entre les trous percés dans la prospection géologique d'ingénierie. Nos premiers résultats prouvent que, en utilisant cette nouvelle méthode, nous pouvons augmenter l'espacement entre les trous percés à 2-3 fois en comparaison avec l'usage de la méthode de pseudo-vitesse. Par exemple pour sonder une grotte dont le diamètre est de 2 m environ, l'espacement entre les trous percés peut être à 30 m environ, ainsi, le travail de forage est de beaucoup économisé. Selon la méthode, on emploie une large fréquence pour émission et une fréquence sélective pour réception, afin de trouver la régularité des changements relatifs de l'atténuation de l'amplitude des ondes de la
simple fréquence dans la masse de roche et de terre. Les instruments et équipements utilisés sont simples et faciles à manœuvrer. Pas de conditions spéciales et indispensables pour faire l'essai. Il est commode de populariser cette méthode in situ. Notre méthode peut être employée dans la prospection géologique d'ingénierie pour sonder les grottes souterraines dont on n'arrive pas à s'informer seulement avec le forage.

1. Introduction

There is a wide distribution of underground grottoes throughout China. Long-lived mining and other human digging activities have left many excavated cavities in small mines and ancient tombs. Also karst caves spread out all over the country. Especially in the limestone areas on the south of the Yangtze River, the grottoes in the overlying strata of the Kainozoic Era are developing very well. All these grottoes have brought about a lot of difficulties in the city, factory and highway construction, damaging the environment. In recent years large-scale mining and pumping of a great amount of groundwater have resulted in a big descent of the groundwater level and caused grotto caving often followed by destruction of buildings, highways and farmland. So an important problem facing the foundation survey and situational engineering geological exploration is to find out the distribution of underground grottoes.

For many years researchers both at home and abroad have undertaken experimental studies of detecting the underground grottoes by means of various geophysical methods. The results show that each method has its unique applicable condition. Take radio wave detecting method for example. The attenuation of radio wave in the medium is related to the electrical conductivity of the medium. The radio wave can pass as deeply as hundreds of meters in through the limestone and dolomite with high resistivity while it passes only a few meters through the water-bearing and loose sediments of the Quaternary System. Therefore this method is difficult to apply. Drilling method needs extremely small borehole spacing which sometimes is reduced to 1.5 m. Obviously it is not economical. As an ideal geophysical method, the crosshole method has already been applied in the engineering survey to study the propagation properties of the elastic wave (including wave velocity, amplitude and frequency) in the rock and soil. In detecting the underground grottoes it has two ways, i.e. pseudo-velocity method (time-difference method) and amplitude attenuation method. On the former one experimental research has been made in China. Fig. 1 is the sketch of the pseudo-velocity method. According to Pernet's principle wave always reaches $S$ along the path where it takes the least time. When there is no transmitting hole receiving hole

\[ V_p = \frac{L}{t} \]

Fig. 1: Pseudo-velocity method sketch

grotto between boreholes the measured pseudo-velocity is $V_p = L/t$. When grottoes exist along the diffraction path as shown simplified in Fig. 1, a formula of the increment of time $(\Delta t)$ can be approximately derived by formula

\[ \Delta t = \frac{\sqrt{D^2 + D'^2} - L}{V_p} \quad (1) \]

where $V_p$ — wave velocity of the surrounding rock, other parameters can be seen in Fig. 1.

The relative increment of time $(\Delta t/t)$ caused by existence of grotto is

\[ \Delta t/t = \sqrt{1 + (D/L)^2} - 1 \quad (2) \]

From Eq. 2 it is known that value of $\Delta t/t$ is related to grotto diameter $D$ and borehole spacing $L$. In Tab. 1 listed are the values of $\Delta t/t$ calculated from different $L/D$ values. Fig. 2 shows the curve of theoretical relation between $D/L$ and $\Delta t/t$ and the measured data. It can be seen from the experiments that the measured $\Delta t/t$ is often greater than the theoretical one. This is due to the wave velocity increasing in the loosening areas caused by the loosened adjacent rock around the grotto.
From Tab. 1 it is seen that when L/D is 5, the value of Δt/t can only reach 2% due to the grotto. Elastic wave measurement has shown that even in the homogeneous medium such as steel, artificial glass etc., the maximum error in observation for the wave velocity measurement may go up to 1.9%. For the limestone specimen measurement the error in observation is generally over 2%. The homogeneity of rock and soil mass greatly exceeds that of rock specimen.

![Diagram](image)

**Fig. 2**: Relation of grotto diameter versus time increment

Hence it is difficult to ascertain the cause in the field for Δt/t being less than 2%. On the other hand the adjacent rock loosing causes the wave velocity descent in the rock and soil mass within a certain scope and Δt/t increasing, which helps the detection of the grottoes. Experience has showed the maximum borehole spacing can not exceed 5 times the diameter of the detected grotto. The low accuracy and small borehole spacing of the pseudo-velocity method makes the popularization rather difficult. The amplitude attenuation method to be dealt with here, however, has features of distinct demonstration, high accuracy and large borehole spacing for detecting underground grottoes.

2. Relatively changing amplitude attenuation method for detecting underground grottoes

2.1 Method and principle

This method is shown as in Fig. 3, A0, A and A1 are relative amplitude values. During elastic wave propagation in the medium the loss of energy causes amplitude attenuation consisting of geometrical attenuation and physical one.

A) Geometrical attenuation

Concerning the elastic wave excited by the point vibration source, as its wave front spreads its energy diffuses also. The geometrical attenuation is related to the wave propagation distance and not to the properties of the rock and soil media.

B) Physical attenuation

During the wave propagation process the particles of rock and soil media are vibrated. The internal friction between grains and between structural planes makes part of energy be turned to heat and lose. Also the structural planes can bring about wave reflection, refraction, scattering and wave pattern conversion, thus increasing energy loss and causing the amplitude attenuation which is named as physical attenuation having a close relation with the properties of the rock and soil media. Experimental data have showed the amplitude attenuation increases with the increase of frequency in some rock with higher moisture.

![Diagram](image)

**Fig. 3**: Amplitude attenuation method sketch

Assume rock and soil mass to be homogeneous medium. If there is no grotto between two boreholes the wave propagates from the emitting point F to the receiving point S and due to energy loss the amplitude decreases from A0 to A. When the transmitted energy is certain and the propagation distance unchanged, A is constant. If there is a grotto between boreholes the wave may first arrive at the receiving point along one of the following two paths:

A) Taking a round course, through A or D
and arriving at S' (F'-A'-S' or F'-D-S'). The amplitude attenuation increases due to the following causes:

a. Propagation distance increases;
b. Adjacent rock is broken more seriously;
c. Reflection and scattering occur at the grotto circumference.

B) Arriving at S' through the grotto (F'-B-C-S').

When the wave passes through the grotto, due to the difference of wave impedances between stuffings in grotto and the adjacent rock, the amplitude attenuation is in relation to the penetrating coefficient \( \frac{x_2-x_1}{x_2+x_1} \) at B and C. The larger the impedance difference between both sides of the interface, the greater the attenuation becomes. In addition to this the adjacent rock loosening helps increase attenuation.

Arriving at the receiving point through either of the above paths, the wave has a remarkable smaller amplitude than without grotto. After measuring amplitudes at points, one by one along the depth of the borehole according to the crosshole method, we can find out the underground grottoes in the light of the unusual amplitude ratio \( A_1/A \).

2.2 Lab experimental study

The experimental study aims at making clear the effect of stuffings and the ratio of borehole spacing and grotto diameter on the amplitude attenuation method. It was done in a water case. Type SYC-2 Acoustic Apparatus for determining rock parameters is applied. Tubular transducers are used as transmitting and receiving probes. The detecting frequency is 35 kHz, water's \( V_p = 1439 \text{ m/s} \), wave length \( \lambda = 4 \text{ cm} \). The experiment array is shown in Fig. 4. The objects to be detected are steel pipes, lead cube, glass bottle, plastic bottle and rubber pipe etc.

In order to eliminate the effect of directivity, the position and direction of the transducers were fixed in the process of the experiment while the objects were moved for measurement. At the same time the transmitting power and the gain of the receiving system were also kept constant. The received amplitude was directly read out on oscilloscope and the propagating time was demonstrated on Nixie light (1 μs accuracy). Thus the transmitted amplitude can be regarded as constant and the received amplitude, without object in water, is \( A \). Adjust the gain so as to make \( A = 20 \text{ mm} \). The received amplitude, if with object between two transducers, is \( A_1 \). So \( A_1/A \) or \( \Delta A/A \) (\( \Delta A = A - A_1 \)) represents the relatively changing amplitude attenuation.

**Fig. 4: Lab simulation experiment array**

2.2.1 Effect of objects with different impedance on \( A_1/A \)

The effect of grotto stuffings (air, water and soil) on detecting is, in essence, represented by different wave impedances. From Tab. 2 it is known that steel wave impedance is 3.7 times the glass wave impedance, so the measured amplitude ratio \( A_1/A \) is nearly less than half of the latter one. This means that the greater the wave impedance difference between the adjacent rock and the Stuffing, the more obvious the unusual phenomenon although the wave propagating path is quite complicated when with object between two transducers. The data in Tab. 3 further show the stuffing effect. Water in grotto can decrease the relative change of the amplitude attenuation.

**Tab. 2**

<table>
<thead>
<tr>
<th>object (cm)</th>
<th>wave impedance</th>
<th>amplitude</th>
<th>remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>steel pipe</td>
<td>6100</td>
<td>7.8</td>
<td>48000</td>
</tr>
<tr>
<td>glass bottle</td>
<td>9500</td>
<td>2.45</td>
<td>13000</td>
</tr>
</tbody>
</table>
| water       | 1439 (t/2)     | 11        | 35000   | 1

2.2.2 Relationship of \( L/D \) versus \( A/A \)

The measured results of objects with different outer diameters are shown in Tab. 3. \( A/A \) has a trend of increase with the decrease of \( L/D \). When \( L/D \) of the lead
cube decreases by half, ∆A/A increases from 10% to 30%. This illustrates that when L/D increases the propagating path becomes longer and Fresnel zone near the object larger so as to make amplitude attenuation more unusual.

Experiment results in Tab. 4 and Fig. 5 further show that for the same object with constant L/D, ∆A/A becomes apparently more increased than Δt/t. Take lead cube as an example, when L/D = 10, Δt/t is only 0.4% and ∆A/A reaches up to 30%. Fig. 5 shows the results of steel pipes at different positions. When L/D = 10, all ∆A/A are 70-75% while the corresponding Δt/t is only 5% or so. The abnormality detected by this method is ten times greater than that detected by pseudo-velocity method, which fully displays the advantage of this method.

2.3 In situ tests

From experience we have known that it is very difficult to determine the absolute wave amplitude. This method measures the relative amplitude so as to eliminate the effect of receiving system. In order to make this method applicable in situ detecting, the following measures were taken:

A) Applying broad frequency emission, selective frequency receiving so as to study the attenuation characteristics of the

---

**Tab 3**

<table>
<thead>
<tr>
<th>Object</th>
<th>Plastic bottle (Φ=3cm)</th>
<th>Glass bottle (Φ=6cm)</th>
<th>Rubber tube (Φ=10cm)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Filled with water</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Amplitude measurement</td>
<td>A1 (μm)</td>
<td>7 19 13 14 15 18</td>
<td>7 19 13 15 18 18</td>
<td>Water: 20°C, f = 55kHz</td>
</tr>
<tr>
<td></td>
<td>A2/L (%)</td>
<td>35 95 65 70 65 90</td>
<td>35 95 65 70 65 90</td>
<td>L = 80cm</td>
</tr>
</tbody>
</table>

---

**Tab 4**

<table>
<thead>
<tr>
<th>Object</th>
<th>Transmitting amplitude line measurement</th>
<th>Transmitted amplitude line measurement</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lead cube (Vp=57000m/s)</td>
<td>4 20 550 0.2 18 10</td>
<td>t1 at/t (s)</td>
<td>4 10 350 0.4 14 50</td>
</tr>
<tr>
<td>Plastic bottle (Vp=27000m/s)</td>
<td>6 10 549 0.9 7 60</td>
<td>A4/A</td>
<td>6 10 549 0.9 7 60</td>
</tr>
</tbody>
</table>

Note: Only integers are used.

---

**Fig. 5:** Δt/t and ΔA/A of object at different positions

**Fig. 6:** Transmitting and receiving system of in situ test
control unit, DC charger, measurement display, air-gap switch and discharge mesh circuit.

The receiving system is made of probes, selective amplifier and optical oscilloscope etc. The probe is accelerometer type with preamplifier. In order to keep a certain coupling condition it has spring clip and has also a higher receiving sensitivity for 1 KHz frequency. \(A_1\), \(A_0\) and \(t_0\) are recorded in the process.

![Block diagram](image)

Fig. 7: Block diagram “HL-1 Electric Spark Seismic Source”

The whole system is automatically controlled. Once the electric spark discharging unit and receiving probe are respectively put into test depths and control signal is given out, the whole process of charging, blasting and recording are completed automatically. In situ operation is quite simple and penetration may reach 40-50 m in hard clay layer and seriously weathered zone.

For determining the exact position of grottoes, the cross transmission method may be coordinated. It is unnecessary to describe this method here because many papers about it have already been published.

3. Example of detecting

3.1 Tunnel detecting in Xiaoshu clay

The test site is located in the 3rd Grade terrace of the Yangtze River. Xiaoshu clay layer is 15-20 m in thickness, including clayey soil above 7 m, clay below 12 m and interbedded clay and clayey soil between 7-12 m. Pure ratio is \(e = 0.7\). The soil is in hard and dry state. The object for detecting is a tunnel of which the floor is 7 m deep under ground surface and the height in 2.5 m.

The profile line is normal to the tunnel axis, borehole spacing is 26 m and the depth is 15 m. The hole diameter \(\phi = 110\) mm. The casing pipe is used for protecting the borehole and sand is stuffed between casing pipe and borehole wall. Water is filled inside the casing pipe during test.

The array is as Fig.6 shows. One measurement was done at 1 m intervals along the borehole depth. Two measurement data are given in Fig.8. The results are rather consistent with each other. \(A_1/A_0\) between two measuring points over and below the floor have distinct difference, the ratio is 3.4. But under the same condition the pseudo-velocity ratio of these two points is only 1.14, showing the advantage of amplitude attenuation method. The data also illustrate this method has a good response to the loosening zone on the top of the tunnel.

![Tunnel test in the Xiaoshu clay](image)

Fig. 8: Detecting tunnel test in the Xiaoshu clay

3.2 Slant hole detecting in the seriously weathered zone of sand shale

The test site is located in a mountain slope, Diluvium 5-3 m deep is on ground surface, below which are Palaeozoic weathered sandstone and shale. There is no groundwater at test depth. The axis of the slant hole is perpendicular to stratigraphical strike and the slant hole height is 3 m.

Test site array is shown in Fig.9. Borehole spacing is 30.5 m in L-1 profile, 47 m in \(X-X'\) profile, hole diameter \(\phi = 110\) mm. The casing pipe was used for protection. Sand is stuffed between hole wall and the casing pipe and water is filled inside the casing pipe during the test. Instrumentation arrangement and test method are the same as in the 1st case.

Part of the measured data are listed in Fig.9 and Tab.5. The results show that \(A_1/A_0\) decreases with the increase of \(L/D\). In L-1 profile, \(L/D = 10\). The minimum \(A_1/A_0\) at slant hole elevation is only 20% - 25% of \(A_1/A_0\) below the slant hole floor.

VIII.296
In profile II-II', L/D = 15, the minimum $A_1/A_0$ at slant hole elevation is 50% of $A_1/A_0$ below the floor.

It is also shown that the measurement difference of pseudo-velocity under the same condition is 3-5%. The curve $A_1/A_0 = F(z)$ in I-I' profile in Fig. 9(b) illustrates the abnormality of caving funnel too.

4. Conclusions

1) Crosshole seismic method is used for detecting underground grottoes apply with the aid of relatively changing amplitude attenuation. When borehole spacing is 10-15 times the grotto diameter, the distinct response to abnormality can be obtained. The borehole spacing can be 2-3 times wider than in pseudo-velocity method. For detecting grotto with a diameter of 2 m or so, the borehole spacing can be limited 30 m or so.

2) Applying the relative amplitude measurement, the instruments and device are simple and operation is convenient. Broad frequency emission and selective frequency receiving can eliminate the frequency effect on amplitude attenuation as well as keep great powerful emission easily. Using MH-1 electric spark seismic source, when frequency is 1 kHz, penetration can reach 40-50 m in hard clay and badly weathered zone.

3) While the detecting line intersects the stratigraphical strike, the parallel-to-bedding planes measurement method can be applied. In combination with pseudo-velocity change, a comprehensive interpretation can be made.

4) Sticked-to-wall probe is able to keep a certain coupling condition. In or-
Abnormality comparison of $V_p, A_i/A_0$ in slant hole detecting

<table>
<thead>
<tr>
<th>Elevation</th>
<th>Wave velocity $V_p$ (m/s)</th>
<th>Amplitude $A_i$ (mm)</th>
<th>Amplitude $A_0$ (mm)</th>
<th>$A_i/A_0$</th>
<th>Wave velocity $V_p$ (m/s)</th>
<th>Amplitude $A_i$ (mm)</th>
<th>Amplitude $A_0$ (mm)</th>
<th>$A_i/A_0$</th>
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<tbody>
<tr>
<td>36</td>
<td>2148</td>
<td>13.5</td>
<td>10.5</td>
<td>1.29</td>
<td>2186</td>
<td>13.0</td>
<td>17.50</td>
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<tr>
<td>35</td>
<td>2259</td>
<td>23.3</td>
<td>17.8</td>
<td>1.32</td>
<td>2362</td>
<td>12.0</td>
<td>16.90</td>
<td>0.71</td>
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<tr>
<td>34</td>
<td>2440</td>
<td>18.0</td>
<td>20.7</td>
<td>0.87</td>
<td>2386</td>
<td>6.5</td>
<td>16.50</td>
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<tr>
<td>33</td>
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<td>13.7</td>
<td>0.58</td>
<td>2327</td>
<td>9.3</td>
<td>16.80</td>
<td>0.55</td>
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<tr>
<td>32</td>
<td>2629</td>
<td>8.4</td>
<td>19.0</td>
<td>0.44</td>
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<td>15.5</td>
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<tr>
<td>31</td>
<td>2652</td>
<td>12.2</td>
<td>6.0</td>
<td>2.03</td>
<td>2281</td>
<td>14.7</td>
<td>18.10</td>
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<td>2926</td>
<td>29.0</td>
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<td>1.10</td>
<td>2283</td>
<td>19.1</td>
<td>17.50</td>
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<tr>
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<td>38.0</td>
<td>14.0</td>
<td>2.41</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ \bar{x}_1 = 2652, \bar{x}_i = 2523, \bar{x}_1/\bar{x}_i = 0.95 \]

\[ \bar{A}_1 = 2289, \bar{A}_i = 2366, \bar{A}_1/\bar{A}_i = 0.56 \]

* $\bar{x}_1$-average below the floor
* $\bar{x}_i$-average inside the hole
* In the table those inside the dashed-lined frame are the measuring points through the slant hole.

In order to increase the measurement accuracy a certain directivity should be kept as much as possible.

5) This method is also effective for detecting the caved-in grottoes.

Like other geophysical methods, what effect this method can produce depends on the study of the object and the investigation of other geological conditions. This method can be regarded as a special method for detecting the underground grottoes which can not be found out only by drilling in engineering exploration.

The author is obliged to electrical engineers Lin Zhongjun and Lin Qinghui, who assisted him in carrying out the experiments.
Theme 7

HISTORY AND DEVELOPMENT OF ENGINEERING GEOLOGY
ABSTRACT

The author gives the history of the creation of the International Association of Engineering Geology (I.A.E.G.) at the XXIIInd International Geological Congress in New Delhi, December 1964. He indicates the main steps of its development from that date on.

1. Naissance de l'Association - New Delhi,
   21 Décembre 1964

La première initiative revient à Asher Shadmon (Israël) qui, le 16.12.1964 au cours du XXIIe Congrès Géologique International dans le Vigyan Bhavan de New Delhi, suscita une toute première réunion sur un sujet qui lui tenait à cœur : les matériaux de carrière et autres produits minéraux utilisés en "engineering".

Bien que surpassant les matières premières minérales métalliques en valeur et en tonnage, ils restaient les grands absents des Congrès Géologiques internationaux. A. Shadmon souhaitait que l'Union Internationale des Sciences Géologiques (IUGS) crée et finance une Commission Internationale qui leur soit consacrée.

Dès cette première réunion, le petit groupe initial (A. Shadmon et Y.L. Picard, Israël ; F. Ahmad, R.S. Chaturvedi, R.S. Mithal, L.S. Srivastava, Indes ; S. Carey, Tasmanie ; K. Erguvanli, Turquie ; G. Bain, USA) estima que la Commission souhaitée couvre également les relations entre les matériaux naturels en place et les travaux de l'ingénieur. Si bien que le même jour, à 16 heures, G. Bain, S. Carey, A. Hamzali, R.S. Mithal, M.S. Balasundaram, S. Krishnaswamy (Indes), A. Shadmon, K. Erguvanli, proposaient une Commission de Géologie de l'Ingénieur de l'IUGS divisée en deux sous-commissions :

1 - Relations des roches en place avec les travaux de l'ingénieur,
2 - Matériaux rocheux utilisés dans les travaux de l'ingénieur.

Des divisions régionales étaient envisagées selon les zones climatiques : tropicale, tempérée, aride, périglaciaire.

Le lendemain 17.12.1964, à 16 heures,
ce projet fut soumis à un plus grand nombre de délégués du XXIIe congrès géologique international, intéressés à la géologie de l’ingénieur, qui avaient pu être prévenus. La motion suivante y fut votée à l’unanimité : "Il est recommandé qu’une commission d’"engineering geology" distinctive soi établie dans le cadre des congrès géologiques internationaux de l’Union Internationale des Sciences Géologiques. Avec cet objectif en vue, des propositions générales sont à soumettre pour examen au Conseil du XXIIe congrès géologique international et de l’IUGS.

La commission d’"engineering geology" serait constituée comme suit :

1) Sous-commission concernant les réactions des roches aux travaux de l’ingénieur

2) Sous-commission pour les matériaux rocheux utilisés dans les travaux de l’ingénieur

3) Sous-commission pour les méthodes et les techniques d’exploration dans les projets de l’ingénieur.

La sous-commission (1) inclurait :
(a) matériaux consolidés
(b) matériaux meubles
(c) facteurs hydrologiques et hydrauliques

La sous-commission (2) inclurait :
(a) pierres de taille
(b) concassés
(c) sables et graviers naturels
(d) terres naturelles
(e) matériaux fabriqués.

Les objectifs de la commission et de ses sous-commissions seraient de promouvoir la connaissance et la diffusion d’informations pertinentes, rassembler des "cas-histories", compiler les bibliographies et catalogues intéressants, fournir les informations sur les recherches réalisées ou en cours, réunir les données géologiques statistiques sur les industries et dresser la liste des recherches nécessaires.

Les travaux de cette commission et de ses sous-commissions seraient groupés au égard au climat, comme suit :

1) zone tropicale
2) zone tempérée
3) zone aride
4) zone pergiglaciaire".

Le samedi 19, le comité exécutif de l’IUGS fit un accueil réservé à la proposition. L’intérêt de la géologie de l’ingénieur n’était pas en cause, mais les faibles moyens de l’IUGS ne lui permettaient pas de prendre en charge une nouvelle commission permanente. Il décida de créer un petit comité : A. Shadmon, président ; L. Calember, secrétaire ; G.W. Bain, V.S. Krishnaswamy plus un représentant de l’URSS à nommer, membres) chargé de dresser l’état de la situation en prenant contact avec les sociétés internationales de géologie de l’ingénieur existantes et de présenter un nouveau rapport sur ce sujet à la réunion du comité exécutif de l’IUGS. (Un rapport devait être effectivement présenté par le Prof. Calember mais la commission de l’IUGS ne fut pas créée. A titre d’anecdote, l’annonce de la fondation d’une association internationale de géologie de l’ingénieur faite par L. Calember à la Société internationale de mécanique des sols conduisit cette dernière à modifier ses statuts pour inclure parmi ses objectifs "les applications au génie civil de la géologie et de la mécanique des roches, de la neige et de la glace").

Dès le lundi 21.12.1964, à 9 heures 30, A. Shadmon rendit compte de ces décisions à une assemblée. Elles furent jugées décevantes. C’est alors que les membres présents déclarent de tenir aussitôt une nouvelle séance qui vit la création, à l’unanimité, d’une association internationale de géologie de l’ingénieur. Ce fut l’assemblée fonda trice de notre association. Elle réunissait : Arnould M., France ; Bain G., USA ; Balasundaram M.S., Inde ; Calember L.M., Belgique ; Chaturvedi R.S., Inde ; Chowdhary G.C., Inde ; Enzo Bene, Italie ; Erguvanli K., Turquie ; Hamza Ali, Inde ; Jain M.S., Inde ; Kent L.E., Afrique du Sud ; Krishnaswamy V.S., Inde ; Lakshmanan J.D., France ; Mahendrâ A.R., Inde ; Manfredini M., Italie ; Prasad V., Inde ; Ramachandra B., Inde ; Rosenqvist J. Th., Norvège ; Sanatkumar Basu, Inde, Shadmon A., Israël ; Srihvaasan P.B., Inde ; Srivastava L.S., Inde ; Zapata M., Espagne.


Le même jour, à 16 heures, dans une réunion restreinte le secrétaire général de l’IUGS indiquait les principales conditions.
que devait remplir l'Association pour être affiliée à l'IUGS, qui préférait beaucoup cette formule et la soutenait ouvertement.

2. Les premiers statuts : Janvier 1967

Les deux années qui suivirent la fondation furent consacrées :
- à compléter le Comité provisoire selon le mandat reçu, afin de lui donner une véritable représentativité internationale. Fin 1966 il comprenait, outre les membres déjà cités : V.S. Krishnaswamy, Indes ; A.M. Hull, USA, en sa qualité de Président de l'Association of Engineering Geologists des USA mais à titre personnel ; E.M. Sergeev et N.M. Kolomenskij, URSS ; Q. Zaruba, Tchécoslovaquie ; M.D. Ruiz, Brésil ; enfin G. Champetier de Ribes, France, coopté pour les fonctions de Trésorier. Des négociations restaient en cours avec l'Australie, le Japon, le Mexique ;
- à préparer des statuts, en conformité avec les règles générales de l'IUGS et conformes aux désiderata des divers membres du Comité provisoire ;
- à préparer un programme d'activités.

L'auteur de ces lignes tient à souigner l'appui particulièrement chaleureux reçu dès l'origine de cette période préparatoire de : M. Martini, Président du Service Géologique fédéral de l'Allemagne de l'Ouest, prématurément disparu ; de l'Académicien E.M. Sergeev, chef de file des ingénieurs-géologues d'URSS, actuel Président de l'Association et de la Division des Sciences de la Terre de l'Unesco, en la personne de son chef, E. Walter.

C'est ainsi que la première réunion exécutive du Comité provisoire s'est tenue à Paris au Palais de l'Unesco, à son invitation et avec son aide, du 9 au 12.1.1967.

A cette réunion participaient : A. Shadmon, M. Arnould, L. Celembert, R.S. Mithal, G.W. Bain et M.G. Green représentant A. Hull, ainsi que E. Walter, Unesco. Des contributions écrites avaient été adressées par : Hull, USA ; Gorski, Sergeev, Ziangirov, URSS ; Balasundaram, Krishnaswamy, Indes ; Erguvanli, Turquie ; Beneo, Italie.

La principale décision a été l'adoption des premiers statuts (1).

La philosophie générale des statuts d'Associations scientifiques non gouvernementales (notamment celles affiliées à l'Union Internationale des Unions Scientifiques ICUS, dont l'IUGS est la branche Sciences de la Terre) repose sur une très large ouverture indépendamment de toutes considérations philosophiques ou politiques.

Toute personne intéressée doit pouvoir adhérer. Dans ce but, l'adhésion directe de membres individuels doit toujours être possible. Pour des raisons d'efficacité,


Cependant, le regroupement en Associations nationales est nécessaire. Mais, même quand elles existent, la possibilité d'adhésion individuelle directe doit rester offerte.

L'objet et les buts de l'AIGI étaient ainsi définis : "Art. 1 : La Géologie de l'Ingénieur couvre les applications des Sciences de la Terre à l'art de l'Ingénieur, spécialement pour les problèmes de l'aménagement du territoire, pour l'art de construire et pour la prospection, l'extraction et l'élaboration des matières premières minérales qu'elle rapportent à ces domaines".

"Art. 2 : Les buts de l'Association sont d'encourager la recherche, l'enseignement et la diffusion des connaissances en développant la coopération internationale dans le domaine de la Géologie de l'Ingénieur".

Un premier programme d'activité fut également adopté, y compris la création de deux "groupes de travail" consacrés aux :
- Pierres de construction et ornementales (Prof. G. Bain, USA)
- Propriétés des roches (Prof. Mithal, Inde).

Conformément aux résolutions, la demande d'affiliation de l'AIGI à l'IUGS a été présentée à la réunion de son Comité Exécutif de 1967 qui l'a acceptée. Cette décision devait être ratifiée par l'Assemblée Générale de l'IUGS à l'unanimité le 23 août 1968 à Prague.

3. La première Assemblée Générale - Prague, 23 août 1968, et sa préparation

La période suivante fut marquée par l'intervention des collègues tchécoslovaques avec notamment pour leader l'Académicien Quido Zaruba et M. Jarošlav Pasek. Responsables de l'organisation d'une section de Géologie de l'Ingénieur au XXIIIe Congrès Géologique International de Prague, 1968, ils partagèrent complètement les idées de

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l'AIIGI. Ils eurent à honneur d'en aider la croissance au maximum de leurs moyens.

C'est ainsi que le premier symposium scientifique de l'AIIGI fut organisé par leurs soins à Brno (Tchécoslovaquie), du 26 au 27 avril 1968, sur le thème "Voies d'injection et parois souterraines dans la Construction hydrotechnique", et un second à l'occasion du Congrès lui-même sur la "Géologie de l'Ingénieur et l'Aménagement du Territoire".

La première Assemblée Générale s'est tenue le 23 Août 1968 à Prague durant le XXIIIème Congrès Géologique International. Comme ce congrès, elle fut gravement affectée par l'entrée des troupes soviétiques en Tchécoslovaquie. Beaucoup de délégués étrangers avaient abandonné le Congrès. L'Assemblée put malgré tout se tenir. Les statuts furent ratifiés et un Comité Exécutif élu pour 4 ans en remplacement du Comité provisoire : Président : Prof. Zaruba Quido, Tchécoslovaquie ; Secrétaire Général : Prof. Arnaud Marcel, France ; Trésorier : Mr. Chemetier de Ribes Gérard, France ; Ancien Président : Mr. Shadmon Asher, Israël ; Vice-Présidents : Prof. Calembert Léon, Belgique ; Mr. Cluff Llyod, USA ; Ing. Ruiz Murillo, Brésil ; Mr. Oborn Leslie, Nile Zélande ; Membres : Prof. Kolomenskij N.V., URSS ; Prof. Zolotarjev G.S., URSS ; Dr. Nemcov A., Tchécoslovaquie ; Prof. J. Janjic J., Yougoslavie ; Mr. Glossop R., Royaume-Uni ; Dr. Ducker A., RFA ; Dr. Tamaka Haruo, Japon ; Mr. Crepeau J.M., Canada.

A compter de cette date le Comité Exécutif, ultérieurement transformé en Conseil, s'est régulièrement réuni une fois par an.

Enfin, furent fondés trois nouveaux "Groupes de Travail":
- Gisements de terrain : responsable J. Pasek, Tchécoslovaquie ;
- Roches solubles : responsables Prof. Reuter, Berlin, et Prof. Erguvanli, Istanbul ;
- Cartographie géotechnique : responsable Prof. Matula, Bratislava.

L'organisation d'un Congrès propre de l'AIIGI, en alternance avec les Congrès Géologiques Internationaux, fut également décidée. Les deux manifestations ayant lieu tous les 4 ans, l'AIIGI aurait donc un Congrès avec Assemblée Générale tous les deux ans, et des Symposiums scientifiques les années intermédiaires, le Comité Exécutif étant élu pour 4 ans par l'Assemblée Générale tenure durant le Congrès Géologique International. La succession historique de ces congrès avec assemblées générales est la suivante:
- New Delhi (Indes) XXIIIè CGI 1964
- Prague (Tchécoslovaquie) XXIIIè CGI 1968
- Paris (France) 1er congrès AIGI 1970
- Montréal (Canada) XXIVè CGI 1972
- Sao Paulo (Brésil) 2è congrès AIGI 1974
- Sydney (Australie) XXVè CGI 1976
- Madrid (Espagne) 3è congrès AIGI 1978
- Paris (France) XXVIIè CGI 1980
- New Delhi (Indes) 4è congrès AIGI 1982
- Moscou (URSS) XXVIIIè CGI 1984

Le bulletin de l'AIIGI et le premier congrès de l'AIIGI, Paris 1970


Le premier Congrès international de l'AIIGI, en septembre 1970, à Paris, marquait une nouvelle étape décisive.

Les principales instances étaient en place. Un état était donné. Mais tout restait fragile, lié à l'activité d'un nombre limité de personnes parmi lesquelles je dois en citer deux qui ont beaucoup travaillé dans l'ombre pour l'Association : Mlle Tran Ho Wouf, une secrétaire de l'époque à l'Ecole des Mines qui, en plus de ses fonctions normales, a averti bénévolement le secrétariat de l'AIIGI et le secrètariat d'édition du Bulletin jusqu'en

5. La suite de l'histoire de l'AIGI - Richard Wolters


Parmi les améliorations progressivement apportées, je citerai :

- la décision de procéder à l'élection du Comité Exécutif, non plus à l'occasion du Congrès Géologique International mais lors des Congrès propres de l'AIGI : (selon une initiative de l'Association brésilienne et du Vice-Président pour l'Amérique du Sud : Fernando Paes de Barros), votée au 28ème Congrès de l'AIGI à Sao Paulo, en 1974 ;

- l'élargissement du Comité Exécutif en un Conseil ouvert aux représentants de chacun des groupes nationaux ;

- la transformation des "Groupes de Travail" en "Commissions". Véritables cellules scientifiques permanentes, certaines d'entre elles ont rempli magnifiquement leur fonction et continuent à le faire, malgré la formidable limitation de moyens que constitue le bénévolat ;


Un très fort potentiel de développement subsiste. En termes quantitatifs, l'AIGI peut décupler.

Pour conclure, l'AIGI est une société savante. Du point de vue scientifique quelques remarques me paraissent s'imposer.

Une excellente originalité réside dans l'intégration, faite dès l'origine, des problèmes de matériaux de construction et industriels à la Géologie de l'Ingénieur. Les relations entre Construction et Matériaux de construction, entre ces matériaux et Pétrographie, pour évidentes qu'elles soient, ne sont hélas pas usuelles.

La complémentarité avec la Mécanique des Solss et des Roches s'est concrétisée par une certaine coordination des activités des 3 sociétés internationales intéressées. Une proposition de regroupement des trois en une fédération de géotechnique a cependant été repoussée à l'unanimité du Conseil tenu à Sydney en 1976. Le rapprochement de la Mécanique des Solss et de la Mécanique des Roches est dans la nature des choses. Il n'en va pas de même pour la Géologie de l'Ingénieur, discipline géologique.


Enfin les problèmes modernes d'Environnement, pluridisciplinaires par essence, conduisent à faire jouer à la Géologie de l'Ingénieur un rôle de synthèse pour définir le cadre physique naturel et pour traiter des interactions de l'activité humaine et de ce cadre.
Je voudrais que la dernière idée exprimée soit celle, chère à tous les fondateurs et que Richard Wolters avait faite sienne. Plus qu'une organisation, l'AIGI est un esprit. Esprit d'ouverture et de coopération internationale, neutre mais dynamique, cherchant à développer les relations humaines autant que les échanges scientifiques.
ABSTRACT

The Brazilian Association of Engineering Geologists (ABGE) was founded in São Paulo, Brazil in August, 1972. Actually ABGE has a effective number of 1,800 members and has created two regional nucleus in the States of Minas Gerais and Rio de Janeiro. ABGE represents the National Group of IAEG with 160 members. Development of Geology and Engineering Geology in Brazil is recent, starting with foundation of courses of geology in 1957. The effective participation of Engineering Geology in projects and civil works in Brazil began just after 1960 attending mainly dam sites projects.

After 1970, other areas of engineering have increased the participation of engineering geology like roads, subways, regional planning and mining.

NATIONAL REPORTS ON GROWTH AND ACTIVITIES

1. Founding of the Brazilian Association of Engineering Geology - National Group of IAEG

In 1968, a group of engineers and geologists met in São Paulo and founded the "Paulista" Association of applied Geology (APGA). Until 1972, the group's activities were limited to the State of São Paulo, Brazil.

That year at a meeting held during the 4th, weeklong "Paulista" meeting on applied geology, the group became a nationwide activity and was named the Brazilian Association of Engineering Geology (ABGE).

2. History an Evolution of Engineering Geology in Brazil.

The effective and growing participation of Engineering Geology in projects and civil works in Brazil, has a history of only two decades: since 1960.

Prior to this, the participation of geology in national problems had been limited to certain historic phases summarized as follows:

- Until the end of the 1930's the few civil engineering works in the country had been designed and even implemented by foreign companies. Mining engineers and some foreign geologists participated in such works.
- But in the 1940's the search for strategic minerals, the arrival of new techniques and new methods of search, encouraged the development of a national geology.

During that period, for the first...
time, Brazilian geologists collaborated in engineering works such as the dam site of "Ribeirão as Lages", in the State of Rio de Janeiro, in which Profs. Viktor Leinz and Luciano Jacques de Noraes were responsible for the geological survey. This was followed by the study for the Paulo Afonso hydroelectric plant on the São Francisco river, in which José Alves de Souza, a mining engineer, participated.

Later, a new impulse was given to geology with the construction of the Anchieta highway crossing the "Serra do Mar" mountain range and linking the city of São Paulo with the port of Santos.

The construction of dams in the South of Brazil on the Camaguá and Jacuí rivers was also important.

Without question, the nucleus which fostered the beginning of geology applied to engineering was the organization of the Geology and Petrography section at the IPT-Institute for Technological Research of the State of São Paulo, in 1937.

From this initial nucleus, Eng. Ernesto Pichler became prominent, and for some 20 years, until his death at the Juquiá dam on the Paraná river in 1959, he worked intensively and productively, introducing new techniques and principles in Brazil.

Pichler introduced rock mechanics in Brazil, when he made tests in 1958 on the rock structures encountered at the Paulo Afonso hydroelectric plant located on the São Francisco River in Northeast Brazil.

• The end of the 1950's is characterized by a great number of hydroelectric plants that were planned, designed and constructed, especially in the South Central region of the country, increasing the application of geological knowledge.

One could even say that hydroelectric installations in Brazil, were the most important reason for the development of engineering geology.

• So much so that the 1960's was the start of the consolidation of engineering geology among us, with an intensive use of geological work on the part of consulting engineering companies and heavy contractors.

It should be noted, that even during this period there was foreign participation in the large projects and civil works. The first occurred from 1962-1966, with the constitution of the UN Committee to coordinate energy studies in the South Central region.

This committee carefully surveyed the hydroelectrical potential of this region and, later, of the Southern and Northern regions of Brazil. As a result, a complete methodology for the execution of projects and surveys of the potential of watersheds was acquired by our specialists. An exchange agreement permitted national technicians to visit Canada and the United States for further training.

In 1967, the project for the execution of the São Paulo subway was begun with the assistance of west German experience. This provided a new opportunity for the participation of engineering geology in a large national project.

If the case of the United Nations Committee and of the São Paulo subway were constructive examples of the importation of technology, the same did not occur with various other projects and works, where many imported packages were technically and economically prejudicial to the interests of geology, civil engineering and to the country itself.

3. Activities

The activities during the period 1980-1983, are summarized as follows:

- Boards of Directions
  1980/1981
  President Prof. Nivaldo José Chiossi
  Secretary Geol. Lindolfo Soares
  Treasurer Geol. Hélio Santucci
  1982/1983
  President Prof. Nivaldo José Chiossi
  Secretary Geol. Lindolfo Soares
  Treasurer Geol. João Jerônimo Monticelli

- Members

  A systematic campaign provided a significant increase in the number of new members:
  1978: 900 members
  1982: 1800 members

  Currently the annual dues for an individual member with a title are US$ 23.00 (January 1982).

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A similar campaign is also now underway to increase the number of members in the IAEU.

Regional Groups

The National Group of the ABGE organized regional in the States of Rio de Janeiro and Minas Gerais. There are regional representatives in the other States, providing contact with the National Board of Directors.

Events

Congress, with an average participation of 280 people and 70 technical papers:

Symposia

With an average participation of 320 people and 10 technical papers published:
- 1979 - Urban and Regional Planning Symposium.
- 1979 - Symposium on Natural and Induced Seismicity.
- 1980 - Symposium on the Geological-Geotechnical characteristics of the Amazon Region.

Round Tables

   b) 1980 - Seismic Risk

Technical Meetings

These are periodic meetings held in various States concerning current topics of interest to Engineering Geology. The following meetings were held during 1980 and 1981:
- São Paulo: 14 meetings
- Rio de Janeiro: 5 meetings
- Minas Gerais: 3 meetings
- Brasília: 1 meeting

Translation of Articles

Publications

Four papers were translated

Glossary of Technical Terms with corresponding English and French Terms pertaining to:
- Drilling equipment
- Structural and Tectonics geology
- Hydrogeology

Technical Bulletins

One technical bulletin was published on:
"Permeability tests in soils"

Procedures

- III CBGE - (Congress)
- IV CBGE - (Congress)
- Natural and Induced Seismicity (Symposium)
- Erosion Control I (Symposium)
- Geological and Geotechnical Conditions of the Amazon Region (Symposium)
- Geological and Geotechnical Conditions of the São Paulo sedimentary basin (Symposium)
- Seismic Risks (Meeting)
- Erosion Control II (Symposium)

Information Bulletin

This is a bi-monthly publication for members, with news of general interest and summaries of articles and presentations as well as information to members.

Geological Engineering Instruction

To better inform students of geology and engineering on the objectives and purposes of geological Engineering, ABGE sponsors visits to various universities of different States with view to presenting the most up-to-date topics concerning Brazilian engineering and geology.


Continuation of the campaign to increase the number of members.

Organization of new Regional Group

Intensify contacts at Schools and Universities through specialized presentations about aspects and problems in engineering geology.

Appoint through Regional groups and representatives an associate of the ABGE in each schools and university.

Increase the number of technical meetings to be held in São Paulo, Rio de Janeiro, Minas Gerais as well as in the other States and the Federal District.
Continue and increase the number of technical publications, bulletins, translations and glossaries of technical terms.

Continue the bi-monthly publication of the information bulletin.

Establish Committees similar to that of seismology for: Hydrology, Dams, Tunnels, Roads, Environment, Urban and Regional Planning, Rock Mechanics and General Geology.

Increase contacts with similar societies in other countries, especially in Latin America.

Promote Regional Symposiums

- November 1982 - Symposium in the Northeast Region of Brazil. Topics and location to be defined.

Prepare the Third Brazilian Congress of Engineering Geology in 1983, in the State of Minas Gerais.

Attempt the organization of the Latin American Congress of Engineering Geology.

4. Future Possibilities of Development

Prof. Nivaldo José Chiassi, president of the National Group considers that since Brazil is now passing through a period of crisis as is the whole world, it is necessary to increasingly upgrade the development and application of engineering geology techniques, so as to adequately serve our Brazilian tropical environment and the current economic situation of the country.

It is useless to insist upon contracting specialists from other continents who are often very capable, and sometimes not and who are not acquainted with our physical environment and the socio-economic conditions of Brazil.

Finally it is important to emphasize that although Engineering Geology "per se" was recognized and increasingly employed in the design phase and construction of dams in the decades of 1960/1970, since 1970 other fields of engineering have increased their demand for collaboration from engineering geology.

This occurred in the areas of roads, planning and mineral extraction.

This could be observed in the case of the roadways designed for the South of the country, of the Sao Paulo and Rio de Janeiro subways, of the airport of Manaus in the State of Amazonas of the Galeão airport in the State of Rio de Janeiro, of the Cumbica airport in the State of Sao Paulo, of tunnels, and may more.

More recently, engineering geology contributes in an important way in the planning, land use and land occupancy, mainly in the metropolitan regions.

Finally, the development of mining activities has provided an increasing field of participation for engineering geology.

ABSTRAIT


En réalité, BETI à un nombre effectif de 1800 membres et a créé deux nouveaux régionaux dans les états de Minas Gerais et Rio de Janeiro.

BETI représente le Groupe National de AIGI avec 160 membres. Développement de Géologie et technique de géologie au Brésil est récent, commençant avec fondement des cours de géologie en 1957.

La participation effective de la technique de géologie aux projets et aux travaux civils au Brésil a commencé juste après 1960 s'occupant surtout aux projets des emplacements de barrage. Après 1970, des autres régions de l'art d'ingénieur ont augmenté la participation de technique géologie comme des rues, des passages souterrain, la planification régionale et exploitation minière.
HISTORY AND PROBLEMS OF ENGINEERING GEOLOGY IN BULGARIA

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ABSTRACT
The engineering-geological science in Bulgaria came into being and developed in the years after the Second World War. Within a short period of time more than 2000 dams, thousands of kilometres of roads, tunnels and canals have been built. They have necessitated special investigations. During that period the engineering-geological research organizations and institutes were founded. At present the major engineering-geological problems are related to the dynamic phenomena - landslides, erosion, and abrasion. The engineering-geological science and practice have achieved great success in the construction of structures on subsidence loess terrains, soft soils and karst regions.

ABSTRAIT
La science géologique de l'ingénieur fit son apparition et se développa dans les années après la deuxième guerre mondiale. En peu de temps on a été construit plus de 2000 de barrages, beaucoup de routes, tunnels et canaux sur lesquels on devait faire des études particulières. C'est en ce temps-la qu'on avait organisé des établissements de géologie de l'ingénieur et instituts de recherches scientifiques. A présent les problèmes de géologie de l'ingénieur les plus importants (essentiels) sont liés aux phénomènes géodynamiques: affaissements, érosions, abrasions. On avait réalisé de considérables succès dans le domaine de la géologie de l'ingénieur et la pratique surtout lors de la construction des équipements au-dessus des terrains effondrés de loess, de maigres sols et régions karstiques.
HISTORICAL BACKGROUND

Bulgaria is a country of ancient civilizations which have left traces of building activities. The Thracians knew how to build settlements upon wooden piles, mines, religious underground appurtenances, solid embankments, etc. The Greek colonists of the Vth and VIth c. B.C., and later the Romans and the Byzantines built many strongholds, ports, roads, water-mains, mines, underground evacuation and water-supply structures, etc., in spite of the soft ground bases, karst, seismic and other unfavourable conditions.

This tradition in building was successfully continued by the Bulgarians after the foundation of their state in 681. By the end of the XIVth c. a number of towns, strongholds and fortifications were constructed. The first two Bulgarian capitals - Pliska and Preslav - were built upon subsidence loess which necessitated the use of soil consolidation by wooden piles, wooden cribs, cushions of compacted and stabilized soil, at the same time developing the structure of the foundations. Hundreds of kilometres of well-compacted embankments accompanied by excavations, both having properly chosen slopes, were also constructed.

All that ancient building activity had its ups and downs but more important than anything else was its compliance with the local natural conditions. In 24 A.D. when a big amphitheatre collapsed because of its soft ground base, the Roman senate issued a decree forbidding further large-scale construction work without preliminary exploration. That year may be taken as the birth date of engineering geology (Cornelius Tacitus, 1969).

After a certain standstill during the late Middle Ages, construction work developed considerably in Bulgaria during the first decades of the XXth century. Within that period of time the transport network was generally built up, the town construction enlarged in scale, some new factories, electric power stations, mining enterprises, etc., were constructed. Several civil and mining engineers, aided by geologists, carried out preliminary investigations for that construction.

A real building boom in Bulgaria occurred after the Second World War when scores of industrial, energy and mining projects were realised. The urbanization of the country and modernization of its transport network was under way. Only within 30 years 2156 dams having a total water storage of $7.7 \times 10^3 \text{m}^3$, thousands of kilometres diversion canals, tunnels, etc., were built (Antonov, et al, 1960). A series of engineering-geological problems related to industrial and civil construction on subsidence and very compressible soils, as well as on unstable ones, were solved in spite of the erosion, abrasion, underground waters and high seismic activity.
The geological investigation work for building the first big dams during 1945-1953 was conducted by the eminent Bulgarian geologists prof. Ekim Bonchev, prof. Strashimir Dimitrov, prof. Tzonyo Dimitrov, and others, together with Soviet specialists. At that time several geologists - Boyan Kamenov, Atanas Demirev, Hristo Antonov, and others, qualified in engineering geology in the Soviet Union. In 1950 was founded the Department of Engineering Geology at the State Polytechnic in Sofia, which in 1953 was transferred to the newly-founded Higher Institute of Mining and Geology. The first head of the department was prof. Boyan Kamenov. Since 1953 the department has trained more than 600 diplomaed engineering geologists and hydrogeologists. In 1952 was founded the Section of engineering geology and hydrogeology at the Geological Institute of the Bulgarian Academy of Sciences. Nowadays engineering-geological investigations are being carried out by several specialised departments of some design institutes: Energoproject, Vodocanal-project, Minproject, Vodproject, Transproject, Sofproject, etc., whose activities are well-known in more than 30 countries in Asia, Africa and Latin America.

Research work is being performed at the Geotechnical Laboratory of the Bulgarian Academy of Sciences headed by prof. Minko Minkov, at the Department of Engineering Geology and Hydrogeology headed by prof. Anastas Demirev, at the Department of Soil Mechanics, Foundation and Engineering Geology of the Higher Institute of Architecture and Civil Engineering headed by prof. Gueorgi Stefanoff, and others. The greatest achievements of the Bulgarian engineering geology and its development are reflected in several monographic works and in hundreds of publications issued in Bulgaria and abroad. The Bulgarian Engineering-geological Society is a member of the International Association of Soil Mechanics and Foundation, of the Carpathian-Balkan Geological Association, and in 1981 it was admitted to membership in the International Association of Engineering Geology. It supervises the journal "Engineering Geology and Hydrogeology" issued by the Bulgarian Academy of Sciences where there are publications not only in Bulgarian but also in other international languages.

2. BRIEF INFORMATION ABOUT THE GEOLOGY OF BULGARIA AND PRINCIPLES OF ENGINEERING-GEOLICAL ZONING

The Bulgarian territory is rather small but it comprises many tectonic structures different in composition and style: part of the Moesian Platform; the Balkanidian folded structures; the Rhodope Middle massif; the Kraishtidian structure, etc. (fig.1).

These structures were further complicated during the Tertiary-quaternary sediment grabens and
depressions (Bonchev, 1956).

On the Bulgarian territory there exist a wide range of lithological, petrographic and engineering-geologic rocks and soil varieties: magmatic - granites, syenites, andesites, rhyolites, tuffs; metamorphic - gneisses, schists and marbles; sedimentary - sandstones, limestones, marls, flysch, etc., having different age - from the Archaic to the Quaternary.

For the engineering geology most important are the covering quaternary complexes consisting of alluvial soils, deluvial clays, subsidence loesses and loess-like sediments, liman-lagoon, sea and lake-marsh soft saturated soils.

Parts of the country are subject to neotectonic movements different in sign and intensity, the biggest lift being in the western part of the Rilo-Rhodopian massif and reaching 4-5 mm per annum, while the biggest subsidence is in the Bourgas depression with the quantity of 2-3 mm annually. These movements and exogenic processes have led to the formation of a complex dynamic relief. The plains occupy 34 per cent, the hilly area - 40 per cent, the semi-mountainous one - 14 per cent and the mountains - above 1000 m - 12 per cent. From a geological point of view it is very important that the larger part of the Black Sea coast, the banks of the big rivers and the hilly areas are...
Fig. 2 Engineering-geological zoning of Bulgaria (Kamenov, et al, 1963)

Moesian region - 1 - Lom engineering-geological district; 2 - Loudogorie-Dobroudja engineering geological districts; 3 - Near Black Sea engineering-geological district; 4 - Fore Balkan engineering-geological district; 5 - Staré Planina engineering-geological district; 6 - Sredna Gora engineering geological district; 7 - Intermontane Valleys engineering-geological district; 8 - Upper Thracian engineering-geological district; 9 - Eastern Rhodopian engineering-geological district; 10 - Western high-mountainous engineering-geological district; 11 - Kettles in Rilo-Rhodopian and Kraishtidian regions; 12 - Kraishtidian engineering-geological region.

built up of soils yielding to sliding, abrasion and erosion, while in the plains the prevailing soils are collapsible, very compressive and dangerous from a seismic point of view.

For the engineering geology are important also the abundance of almost all possible kinds of underground water (fresh and therrmonineral), the complex hydrographic net, the changes in the temperature-moisture and rainfall regime of the country. The climate is continental with Mediterranean influence. The earthquakes introduce also a marked dynamic element among the physico-geological and engineering-geological processes and phenomena. The technogenetic factors acquire a further significance.

According to the Map of engineering-geological zoning (Kamenov et al, 1963), the country is subdivided in regions and districts (fig. 2).
The regions are distinguished on the basis of the tectonic macrostructures and the districts - on the basis of the mesostructures, morphology, lithology, character of geodynamic processes and prevailing types of the rock masses. One of the first engineering-geological maps of the scale of 1:500000 in the world is that of Bulgaria (Kamenov et al). Engineering, geological and microseismic zoning has been made in a larger scale for separate parts of the country. A typification scheme has been made up for some soil varieties.

3. MAJOR PROBLEMS OF ENGINEERING GEOLOGY IN BULGARIA

3.1. LANDSLIDES

The landslide problem is the heaviest and so far fully unsolved engineering-geological problem of Bulgaria. The slides are irregularly distributed on the country's territory. They are tied mainly to the Tertiary formations, to the tectonically eroded zones, to the earthquake regions, to the river banks and sea shores in whose composition clay-marly varieties may be found (Kamenov, et al). The biggest slides are in the Rhodopes, along the Black Sea coast and the Danube river banks. An almost continuous stripe of 80 km long and up to 6 km wide has developed from the Black Sea coastal zone north of the town of Varna, They occurred during the Pleistocene as a result of great variations of the sea level reaching up to 150 m which have affected a tectonically disturbed coast. In these places the coast has generally lifted while the sea bottom has sunk because of the load of 800 m quaternary sediments. Heavy earthquakes with high intensity have had a considerable influence. In the 1st c. B.C. such an earthquake destroyed the ancient Greek town of Bizone.

The landslides comprise Sarmatian marls and limestones, the sliding surface passes along konk, lower and middle Sarmatian marls. The lithological differences affect strongly the type of landslides which to the north gradually change to landslides of the block-packet type (Kamenov, et al, 1973). The most typical representative of the latter is the Balchic landslide (fig. 3).

The old Pleistocene slides are now becoming active mainly in their lowest part under the influence of sea abrasion and technogenic factors, the lifting of the water table being the most important one. On the sliding terrains are built some of the biggest and most beautiful Black Sea resorts which have world fame, such as Druzhba, Zlatni Piassatsi, Albena, Roussalka, etc.

The slides to the south of Varna have a more limited distribution, comprising terrains built up of clayey materials, and the sea abrasion is the most important factor of their activation. The most typical representative is the
Sarafovo landslide.

Another area with widely spread big slides is the right bank of the river Danube. In these places since the Pleistocene period the asymmetric river valley has been continually developing to the south and enormous slides have formed mainly in sectors where the high bank is built up of Pliocene clays. The stability of the slope is worsened by the accumulated thick loess covering during the Middle and Upper Pleistocene. The predominant type of slides are the delapsive ones but detrusive slides similar to the one which destroyed the village of Orsoya, are also to be found.

In recent years the Danube bank has been intensively investigated in view of designing large hydrotechnical structures. After their construction great activation of slides is expected. The total length of the Nikopol-Tournu Magurele reservoir is 287 km, of which the high banks are 90 km, 60 km being covered by old and contemporaneous landslides. Several towns and villages are built on big slides - Cryahovo, Ostrov, Dolni and Gorni Vadim, Somovit, Nikopol, Svishtov, Vardim, Toutrakan, etc., which suffer considerable material losses every year.

Numerous are the slides in the interior of the country, in mountainous and valley slopes built up of Pliocene and Paleogene sediments, flysch, marls and clays, steep embankment, and have various character. It is regrettable that their activation has recently been connected with the construction of different appurtenances and the water supply of towns and villages which as a rule gets ahead of the properly built sewerage system.

Big landslides and surface
terrain subsidence occurred during the construction of open-cast and underground mines. Particularly big were the slides in the East-Maritsa lignite basin in 1966 and 1968 (fig. 4), some of which reached up to 75x10^6 m^3 and hampered the normal exploitation of the structures.

A special Trust for fighting against landslides and abrasion was recently founded with headquarters in Varna. Its main task is to carry out investigations, to design and put in practice anti-sliding measures.

In Bulgaria, as in other countries, the fight against slides is very difficult, especially when they have large proportions. There is no reliable methodology yet for evaluation of their stability which should take into account the complex interrelations between various factors. The existing methods are practically applicable for smaller slides.

3.2. THE LOESS PROBLEM

About 15 per cent of the Bulgarian territory and 50 per cent of its most fertile and densely populated areas are covered with subsiding loess. The eolian loess in the Danube Plain is most widely spread, its thickness reaching in some places up to 100 m - one of the thickest loess covers in the world. Here the loess is composed of 6 or 7 horizons divided by buried soils. To the south the loess gradually gets thinner and the loess cover is dismembered (fig. 5). The loess horizons merge with the soils in a pedocomplex.

The loess region in the Bulgarian part of the Lower Danube Plain is one of the best studied in the world. A geologic-lithologic and a thickness and proportion of subsidence maps have been made of that part. The basic relationships in the change of the soil mechanic indexes have been determined, the loess basés have been typified and the most appropriate methods of fighting against subsidence and filtration of various foundations have been worked out. The results
Fig. 5 Total value of the loess-collapsible subsidence at geological load - $\Delta S$ /by M. Minkov, K. Stoilov/

1 - Collapsible subsidence value between 0 and 5 cm;
2 - Collapsible subsidence value up to 25 cm;
3 - Collapsible subsidence value up to 50 cm;
4 - Collapsible subsidence value up to 75 cm;
5 - Collapsible subsidence value up to 100 cm (or more)

obtained have been summarized in three monographic works (Steffanoff and Kremakova, 1960; Minkov, 1968; Minkov and Evstatiev, 1975), in some studies and over 100 publications printed in Bulgaria and other countries.

One of the most peculiar features of the Bulgarian loess is its vertical and horizontal heterogeneity (Minkov and Evstatiev, 1977). Besides the facial changes in the composition and properties of the loess from north to south and from east to west (from the loess-like sands, through the sandy, typical and clayey loess, to the loess-like clays), there are also abrupt changes depending on the macro- and meso-relief features which could cause a longer building to fall upon a loess basis having totally different soil-mechanic indexes.

The relative collapsive settlement under the action of an additional load of 300 kPa for the uppermost loess horizons which are most important for building is as follows: for the loess-like sand - about 2 per cent, for the sandy
loess - 4.5 per cent, for the typical loess - 6.5 per cent, for the clayey loess - 7 per cent, and for the loess clays it is 2 per cent. A number of structures built in the past have been affected by loess settlements. The biggest subsidence was measured at a hydromeliorative canal in a typical loess. At the same time, in some negative forms such as the "steppe pane", under the influence of centuries-old infiltration the loess is recompacted to such an extent that it presents one of the best soils ever existing.

One of the peculiarities in the investigation of the Bulgarian loess soils is their complex character.

Along with lithologic-stratigraphic and mineralogic investigations, intensive soil mechanic studies have been carried out, including representative experiments in situ (Minkov, et al, 1977). In connection with the latter problem in 1975 a special experimental station of the Geotechnical Laboratory at the Bulgarian Academy of Sciences was founded in the town of Rousse.

In the struggle against infiltration from different kinds of water-leveling basins and reservoirs, cement-soil and lime-soil screens stabilized in situ have been used with success. The following methods have been applied against subsidence: compaction with super-heavy tampers (more than 15 tons); hydrocompaction; silication; surficial and deep hydrocompaction; compaction with short pyramidal piles and cement-soil cushions. The latter method was created in Bulgaria and has been so far applied in the construction of scores of buildings and structures including the atomic power station, as well as television towers 180 m high, tall buildings, cooling towers, etc. It has been proved that the stabilization of the loess bases according to the above method has an anti-seismic effect.

An interesting trend in the complex investigation of loess soils is the utilization of waste-products from electric power plants and industry for low construction in the loess region void of natural reserves of sand, gravel and broken rock.

3.3 ABRASION AND EROSION

As was pointed out in 2.1, abrasion is one of the main factors in disturbing the stability of the slopes. The Black Sea coast having a length of 378 km, 271 km or 72 per cent of the whole are subject to abrasion. The average annual quantity of the material carried away is 1344.1 thousand m³. The abrasion rate is usually 1 m per annum, but in some economically important parts this value is between 7 and 8 m (Shouiski and Simeonova, 1982).

The abrasion rate depends on the geomorphologic and lithologic conditions. The southern Black Sea coast is low and built up of solid Upper Cretaceous volcanic and
Eocene sedimentary rocks and there the abrasion is of no practical importance.

Abrasion along the Black Sea coast is for the time being an unsolved problem. Most of the 80 anti-abrasive structures built so far (mainly rock-fill embankments) do not perform their function. Intensive investigation work is being done at present in the sphere of maritime lithodynamics and engineering geology, aiming to put the Bulgarian coast protection activities on a sound footing. Only thus could the budgeted funds for the next few years, being twice the funds used so far, be utilized efficiently.

Expeditionary investigations on the bank conditions of all bigger water reservoirs in Bulgaria have been performed to fulfil a special research program of the Geotechnical Laboratory at the Bulgarian Academy of Sciences. It was determined that for 56 of them abrasion offers some problem and the dams necessitating systemic regime surveillance in relation to building a bank-protecting structure, have been registered.

A certain idea of the significance abrasion has for the Danube bank, was given when the slides were considered. The construction of the hydrotechnical complex "Nikopol-Tournu Magurele" will cause abrupt activation of the abrasive process and in the area of easily washable loess it may lead to the disturbance of a strip of land up to 200 m wide (Anguelov, Kostova, Anguelova). This prognosis is supported by the fact that in pre-historic times the river Danube flowed 50 km to the north and since then it has been continually moving to the present Bulgarian territory as a result of abrasion.

For averting the activation of abrasion resulting from the dam construction, strengthening measures have been envisaged on the high Danube bank sectors.

The surface erosion of the soils is observed in the Fore-Balkan regions. Lately, as a result of complex agrotechnical measures, it has been reduced to a minimum. This also applies to the East Rhodope areas where some of the best brands of tobacco are grown.

Especially big funds have been invested for restricting the river and ravine erosion. The efforts have mainly been directed towards investigations of the upper reaches of the Bulgarian rivers whose catchment area is located in the Fore-Balkan, Rila, Pirin and the Rhodopes.

34SOILS

Besides the loessy soils there are also other types of clays of mainly Quaternary age - sea and river silts, non-compacted alluvial dusty and sandy clays, swelling organic soils, etc. They are spread along the Black Sea coast, the river Danube, in densely populated plains and offer a great problem, especially taking into account the
high seismic activity of the country.

Great attention has recently been paid to the sea and river silts relating to the use of marshy terrains as well as the expansion and modernization of port structures. For the time being the expensive pile foundations are mostly applied, but new ways are sought for using the natural bearing capacity of the soils. For this purpose both the laboratory and field methods have been improved, including the usage of pressiometers, rotary shearing, etc.

The swelling soils in Bulgaria are of two types: clayey smolnitsa - a thick contemporaneous layer having high organic content, and montmorillonite-illite clays of Pliocene, Sarmatian and Paleogene age. They are to be found in the Sofia plain, the districts of Nihailovgrad, Yambol and Pernik, etc. When dried, their volume is reduced to 40 per cent and they crack in depth as a result of which dozens of one- and two-floor buildings have suffered heavy deformations.

The quaternary swelling clays (mainly of Pliocene origin) offer a problem for the tunnels, mines, deep building excavations, open-cast mines and the construction of the Sofia metro.

3.4 THE EARTHQUAKE PROBLEM

For a period of about 120 years 15 damaging to catastrophic earthquakes have been registered in Bulgaria. During the first third of the XX century strong earthquakes occurred in all significant earthquake zones. Besides that the Bulgarian territory has often been damaged by earthquakes having their epicentres in neighbouring countries, as was the case with the Vrancha earthquake which took place in Roumania in 1977 and caused considerable damages and casualties.

After the catastrophic earthquake in Skopie - Yugoslavia in 1963 almost all big seismic foci in Yugoslavia, Greece, Roumania and Turkey were reactivated. During that period the big seismic zones in Bulgaria "kept calm". The earthquakes in the neighbouring countries and especially the Vrancha one, caused attivation of the investigation work in this sphere.

A team headed by acad. Ekim Bonchev have made up prognostic maps of a new type with the probable seismic intensities (fig. 6) on the basis of complex geological and geophysical studies.

At the Geotechnical Laboratory there is a section of dynamic engineering geology where research associates work out the problems concerning the effect of engineering-geological and hydrogeological factors on the seismic risk, on the change of strength and deformation properties of building soils after seismic action, on the importance of earthquakes for the activation of landslides.

During the period of 1980 -
1982 a team of engineering-geologists, hydrogeologists, seismotectonics, seismologists and geophysicists carried out microseismic zoning of the territories of important energy and industrial sites in the Thracian Plain. The social and economic usefulness of that investigation is great and will serve as a methodological basis for the microseismic zoning of important economic regions in the rest part of the country.

4. OTHER ENGINEERING-GEOLOGICAL PROBLEMS IN BULGARIA

Of the hydrogeological phenomena which are important for the building activity we shall point out the raising of the ground water table in the urban areas and irrigation systems with all the resulting unfavourable consequences. Because of these processes the downfalls and slides in the loess terrains have been activated.

The karst in the carbonate rocks is manifested in areas covering 6 per cent of the Bulgarian territory. (Kamenov and Iliiev, 1963) It has caused difficulties for the hydrotechnical and tunnel construction.

Suffosion phenomena are established as single cases in the water-saturated sands along the
Danube river banks and the Black Sea coast.

Talus, downfalls and snowslides are established in the mountainous areas and affect the hydrotechnical, road and tourist construction work.

Many ecological problems arose recently, the solution of which necessitates engineering-geological competence. The enormous massifs of lagooned ashes from the thermoelectric power stations, phosphogypsum and other waste products are commensurable in volume with some soil varieties composed during the Quaternary.

From the aforesaid it may be concluded that various and widely spread phenomena of geodynamic origin are found in Bulgaria.

5 Conclusion

In this paper the authors are not in a position to give an entire picture of all engineering-geological investigations performed in Bulgaria. The attention was mainly focused on the basic geodynamic phenomena and processes. The prospects for the development of engineering geology in Bulgaria are very encouraging for in the near future the building of enormous reservoirs, large industrial-energy complexes, irrigation structures, underground and open-cast mines, etc., is expected. For instance now the total area covered with open-cast mines in the East Maritsa basin is about 50 km², and the output is 19 million tons per annum.

After 1985 it is expected to increase up to 45 million tons annually. This is a rather big figure for Bulgaria, having in mind that the present total cut out is 31 million tons.

Thus for the following generations of engineering geologists there will be a lot of useful work to do. We truly hope that the membership of our country in IABG will contribute to a better exchange of scientific information between the Bulgarian engineering geologists and their colleagues all over the world. That will promote our common cause towards fitting our structures properly in our surroundings.

References


ABSTRACT

The founding of the Japan Society of Engineering Geology goes back to 1955 when five years of original association are included.

Responding to the social needs the Society has developed to have 1515 individual and 125 supporting members in 1981.

The Society is affiliated to the International Association of Engineering Geology through the Science Council of Japan and by organizing International Committee within the Society.

History of organization is given briefly. The Society sent Vice-President of Asia, sends Commission members, deligates to congress/symposium and council meeting of IAEG.

Quarterly Journal is issued from the beginning of the Society and it numbers Vol. 22 in 1982.

Titles of papers for which the Society awards were given are listed.

The tendency of study subjects on engineering geology during the years from establishment of the Society is reviewed.

Finally few of the future implications are briefly stated.

ABSTRACT

L'établissement de la Société Japonaise de Géologie de l'ingénieur retourne à 1955 lorsque cinq ans d'association originale sont inclus.

Répondant aux besoins sociaux, la Société a développé pour avoir 1515 individus et 125 membres supportant en 1981.

La Société est affiliée à l'Association Internationale de Géologie de l'Ingénieur par le Conseil Japonais de Science et par organisant Comité International entre la Société.

L'histoire d'organisation est mentionnée brièvement. La Société a envoyé le Vice-Président d'Asie, envoie des membres de Commission, des délégués au Congrès/banquet et réunion de conseil de SIGI.

Journal trimestriel est publié dès le commencement de la Société et il numérote Vol. 22 en 1982.

Titres des articles pour lesquels les récompensés Sociales étaient distribuées sont cataloguées.

* Compiled by Dr. Toru Onodera, The former president of Japan Society of Engineering Geology.
La tendance des sujets d'études sur géologie de l'ingénieur pendant les années d'établissement de la Société est revisée.

Finalement quelques implications de futur sont brièvement mentionnées.

INAUGURATION

In April 1955 a small session was held on the geology and natural disasters as one of the evening sessions in the 62nd annual meeting of the Geological Society of Japan. The ten attendants discussed the importance of the theme and concluded to organize a group to begin and continue to collect the data on geology on natural disasters. The first liaison bulletin was issued in September 1955.

In April 1957 the theme of the group was enlarged from disaster geology to engineering geology, and the number of members increased to 141 in 1958, to 161 in 1959 and the Journal numbered up to eight by the end of 1959.

ADVANCEMENT

The advancement of the Society can be seen by the increase of members as shown in Fig. 1.

The expansion of the Society can be said to be due to the increase of public works such as dam-, highway-, railway-, irrigation-, erosion control works etc. that are intimately concerned to the earth's crust. At the same time it can be said vice versa that the advancement of engineering geology has contributed to the increase and enlarging of such public works.

Fig. 2. shows the breakdown of the individual members in which it is seen that the number of academic and government institutes has not so much increased. This situation can be regarded as something likened to the base-flow in hydrology.

Fig. 3 shows the percentage breakdown of the transition of individual members. Around these years civil and other works became very active and large scale.

Responding to these social activities engineering geology has become more required and the number of engineers/technicians in private enterprises has also increased year by year to raise the public opinion to establish an organization of information interchange.

Responding to social circumstances the Japan Society of Engineering Geologists was established as a study association in April 1960 with number of individual members 173 and supporting members 20. The first president of the Society was HIROTA, Koichi who passed away in 1972.

The Society had become so consolidated as to change its name to the Society of Engineering Geology of Japan in 1966 with individual members 420 and supporting members 39. In 1979 English name of the Society was changed to Japan Society of Engineering Geology.

Annual fee of the individual member is in converted U.S. dollars about 20. The address of the Society is:

The Japan Society of Engineering Geology, 2-10-11 Nishiwaseda, Shinjuku-ku, Tokyo 160, JAPAN

ORGANIZATION

The past and present Presidents are:

1960 - 1965 HIROTA, Môichi,
1966 - 1968 SAITO, Shoji,
1969 - 1972 TANAKA, Haruo,
1973 - 1977 ONODERA, Toru and
1978 - KURAMOCHI, Fumio

The past and present Vice-Presidents are:

1976 - 1977 IKEDA, Kazuhiko & KURODA, Hi-detaka and
1978 AKUTAGAWA, Shinji & IKEDA, Kazuhiko

From 1968 to 1975 the Society was operated by four committees -- planning, editorial, event and general affairs. From 1975 to 1977 the Society was operated by six committees -- general affairs, editorial, event, planning, international and finance. Since 1978 the Society is operated by four committees -- general (chairman YOSHIKAWA, Keiya, 8 members), editorial (chairman KOJIMA, Keiji, 13 members), event (chairman MIYAJIMA, Keiji, 5 members), and international (chairman NOZAKI, Koji, 6 members).

The Society has three Branches, i.e. in Hokkaido, Kansai and Kyushu respectively.

The Society has three honorary members: TAKADA, Akira, TATEWA, Iwa and SAITO, Shoji and has 1515 individual members and 125 supporting members and the Society is governed by 39 Councilors, 14 Executive Directors and 2 Auditors.

ACTIVITIES

The Society is affiliated to the Science Council of Japan since 1963 and ONODERA, Toru (1963 - 1973) was and KOJIMA, Keiji (1973 - ) is the Chairman of the Subcommittee on Engineering Geology, Committee on
Geology of the Science Council of Japan. The Society is affiliated to the International Association of Engineering Geology as the National Group of Japan by organizing international members within the Society.

TANAKA, Haruo served IAEG as the Vice-President of Asia 1969 - 1978. The Society has sent delegate(s) to every Congress, Symposium and Council Meeting ever since 1970.

ONODERA, Toru served IAEG as a member to the Teaching Commission since 1974.

SUGAWARA Hayashi is serving as a member to the Mapping and other Commission in IAEG. The Society is ready to participate in other Commission.

NISHIKAWA, Yasushi (1963 - 1972) served and YOSHINAKA, Ryunoshin is serving IAEG as its editor in Japan since 1973. The Society issues quarterly journal since 1960 of which papers are mostly in Japanese with English summary. The Society holds normally annual reading meeting to which about twenty papers are presented, and also occasionally holds symposium on specific topic. For instance, the theme of symposium held in October 1980 was "The measuring and evaluation of rock mass containing fault or crush zone" with chairman OKAMOTO, Ryuichi and co-chairman YOSHINAKA Ryunoshin.

Special issue of Faults on Construction Work with 17 papers are given by the Society in 1981.

The Society award is given to the excellent paper which eminently contribute to the promotion of engineering geology. The following papers have been thus far awarded.


Tendency of Study on Engineering Geology during Recent Two Decades

When the subject of study on engineering geology are grouped into the following main items, the trend of study topics changes as shown in Fig. 4 by IKEEDA, Kazuhiko.

1. Topographical/geological survey and geological structure;
2. Weathering, mass movement/land slide, erosion, land subsidence etc. related to preventing geological hazard;
3. Mode of occurrence, permeability, spring, water quality and variation of groundwater;
4. Engineering properties of rock and ground;
5. Survey method such as drilling, geophysical prospecting and logging;
6. In situ tests on strength, deformation, strain, initial stress, permeability etc.;
7. Earthquake, ground vibration, active fault, crustal movement etc. and
8. Others including, volcanic and geothermal phenomena etc.

From Fig. 4 the following remarks are suggested by IKEEDA, Kazuhiko. The greatest number of study subjects is those that are related to disaster prevention (item No. 2). Studies on these subjects are performed in similar amount yearly, and the contents of the studies are substantial year by year. Though the advance of the study is clear, it is unfortunate that there is no presentation on the subject of rock fall and tunnel collapse. As these phenomena often give us damages the study results on these topics are looked forward.

Taking the place of studies on topographical/geological survey and geological structure (item No. 1) of which number of studies per annum decreases ever since ground 1969, number of study results on the engineering properties of rock and ground (item No. 4) are increasing rapidly. This situation is presumably due to contribution to the detailed design and safe execution of large construction works. The study on this item will be continued widely in future also.
Studies on geological survey method (item No. 5) have been carried out almost in similar pace during these two decades, new methods are established by and the survey accuracy has been increased. However, more accuracy is often demanded to respond to the requirement of accuracy of civil engineering or disaster prevention works and the study results to satisfy this requirement are looked forward.

Next, nevertheless papers on in situ test (item 6) are rather few during 1970 to 1975, studies on this subject have every possibility of development in future according to the enlarging of civil engineering structure, the introduction of new construction techniques and to the demand of safe and rapid execution.

Finally, study results on the subjects related to item 7 have begun to be presented from 1971. These subjects, together with the subjects of item 8, will be expanded by the development of new theory and by the social demand. So published paper could be found on the geological problems related to changes of environment during two decades. Even though there might be restriction of publication on the side of enterprise, study results on such environmental geological problems as related to ground vibration earth temperature etc. are looked forward to be made public not to leave greater problem in future.

CONCLUDING REMARKS

The field of engineering geological study has increased and is increasing year after year. Though the weight of the topics has changed along with time, we have increasingly many subjects to study and the study results do have possibility of infinite development.

We have to extend our concern from applied science through engineering science to technology or engineering and moreover to techniques of the earth's crust, that is from engineering geology through geotechnology or geotechnical engineering to geotechnique.

In practical side the Society hopes to take some measures in encouraging those who want to obtain the qualification of nationally authorized consulting engineer and also in bringing up the young people who want to take their ways in engineering geology or geotechnique.

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Fig. 1 Transition of number of members of Japan Society of Engineering Geology
Fig. 2 Breakdown number of individual members of JSEG by occupation

Fig. 3 Percentage breakdown of individual members of JSEG

Fig. 4 Tendency of study subject by IKEDA, Kazuhiko, 1979
ABSTRACT
The events from 1964 to 1982 concerning the affiliation of engineering geological interests in the U.K. are recorded and discussed.

RESUME
Les événements depuis 1964 jusqu'ici concernant l'affiliation des intérêts de géologie de l'ingénieure en le royaume uni sont rapportées et discutées.

INTRODUCTION
It is now a fact of the history of engineering geology that the question of an International Association of Engineering Geology was first mooted in 1964 at the Geological Congress in India (Shadman, 1970). In the next four years much effort was put into laying the foundations of IAG, to such good effect that by January 1967 affiliation to the International Union of Geological Sciences was unanimously accepted following a submission by the IUGS Committee on Engineering Geology (Arnould, 1970). The first General Meeting of the Association was held in Prague in 1968 on the occasion of the International Geological Congress. For the first time at the IGC there was a separate session devoted to engineering geology with 450 registered members. Technical sections followed very successful pre-Congress field excursions.

At the General Assembly in Prague in 1968 an Executive Committee was also elected which included Mr. R. Glossop as the United Kingdom member. Mr. Glossop was, at that time, Chairman of the Engineering Group of the Geological Society of London, and it was to take 7 years before application for the affiliation of engineering geological interests in the United Kingdom to the IAG was made, and accepted by Council, at the Council meeting held in Krefeld in 1975.
It is the purpose of this account to recall the events of those seven long years, and the subsequent relations between U.K. engineering geology interests and the IAE.

Events following the Prague meeting in 1968

The decision to elect a U.K. representative on the Executive Committee was confirmed in September 1968. Mr. Glossop could not accept that the election in Prague was valid, and proposed that the matter should be discussed, without commitment, with the Committee of the Engineering Group of the Geological Society. By February 1969 it had become clear that the question of collaboration between the Engineering Group and the IAE seemed to present some difficulties. Because of the international implications of affiliation of the IAE with IUSS, the matter would have to be taken up with the British National Committee for Geology of the Royal Society.

Before that took place Mr. Glossop attended an Executive Committee meeting of IAE in Paris in May 1969, as an independent observer. It is of interest, having in mind subsequent events, that at that meeting much time was devoted to the question of launching a Journal, and it was suggested that the Geological Society of London might be willing to establish the Society's Quarterly Journal of Engineering Geology as the official Journal of IAE. Although this was eventually agreed to by the Council of the Geological Society of London, a decision had already been taken that an IAE journal would be edited in Prague and printed in Paris.

The discussions were reported to the British National Committee, with emphasis on which organization should become the adhering body should the U.K. become a member of IAE.

Meetings of the Executive Committee of IAE were to be held on a yearly basis, with the next meeting proposed for Paris in September 1970. The intervening 15 months were to bring interesting developments.

The London Section of the Association of Engineering Geologists and IAE

In April 1970 the chairman of the London Section of the (American) Association of Engineering Geologists proposed to the Secretary General of IAE that they should be accepted as the British National Association to be affiliated to IAE. This suggestion caused a flurry of activity. Although the American AEG showed interest in IAE, there was no intention on their part of a closer association in the near future. Some hope was expressed that active interest by the U.K. AEG might stimulate the parent body itself to take an active part in the IAE.

Private discussions made it quite clear that the suggestion that the local section of AEG should officially represent British interests in engineering geology on IAE was undesirable. A different solution would have to be sought. The Engineering Group of the Geological Society was the obvious candidate, but there were administrative problems to sort out. Incidentally, there was no official connection between the British section of AEG and the Geological Society, although there were Fellows of the Society who were members of AEG.
RESOLUTION OF THE PROBLEM OF U.K. OF AFFILIATION TO I.A.E.G.

In October 1969 I became Chairman of the Engineering Group and Mr. Glossop continued as independent member of the Executive Committee of IAEG.

By April 1970 it had become clear that official geological opinion in the U.K. was opposed to British engineering geologists being represented on IAEG through the British Section of the American AG. Late that month Mr. Glossop and I discussed with the Executive Secretary of the Royal Society the problems involved in closer collaboration between the Engineering Group of the Geological Society and the IAEG. Discussion mainly concerned financial support to meet annual membership fees; the response was not discouraging.

The upshot was the preparation of a discussion document (Dearman, 1970) to the Engineering Group Committee and Council of the Geological Society of London setting out the problems associated with the IAEG. These included:

(a) direct collaboration with the Engineering Group acting as the British National Committee to the IAEG. (In May 1969, U.S.S.R., Czechoslovakia, Yugoslavia, F.R. Germany, Finland, Brazil, Ghana, Israel had established National Committees; U.S.A., U.K. and France had not).

(b) membership of various working parties by representatives from, or nominated by, the British National Committee. Working parties have been proposed, or are already in existence to deal with the following topics: Engineering Geology Maps, Soluble Rocks, Mass Movements, The Storage, Processing and Retrieval of Geological Data, and Education. (In May 1969, W.R. Dearman had been suggested as a member of the working party on Engineering Geology Maps, and Dr. J. Hutchinson as a member of the working party on Mass Movements).

(c) The Quarterly Journal of Engineering Geology as the official journal of the IAEG.

As a comment to the document, the minute of the meeting of the British National Committee meeting, held on May 29th 1969, was reproduced. In addition there was a note that:

(d) On Wednesday, 29th April, 1970 Mr. Glossop and I discussed with Dr. Martin at the Royal Society, the problems involved in closer collaboration between the Engineering Group of the Geological Society and the IAEG with, as a final emphasis, the remark that:

(e) At the committee meeting of the Engineering Group to be held on Friday, May 29th, 1970 we should adopt a positive attitude to items (a), (b), (c) set out above, and decide what proposals, if any, the Committee should make to the Council of the Society. Council meets on Wednesday, 3rd June, 1970.

Council of the Geological Society of London duly discussed the matter, and additional notes were prepared for the next Council meeting on June 24, 1970. The following points were made that:

In September 1970, a meeting of the International Association of Engineering Geology is to be held in Paris.

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The following topics are likely to be discussed at this meeting:

(a) Establishment of a British National Committee for Engineering Geology. The representative of the British National Committee would be a member of the General Assembly and might be elected as a member of the Council of IABG.

(If the UK is to have any association with the IABG, then clearly the Engineering Group should be involved. What has to be decided is whether the Engineering Group is truly representative of British engineering geology and so could assume the role of the British National Committee or would some wider grouping of interests be more appropriate).

(b) Representatives from the U.K. on various working parties.

(These should be nominated by the British National Committee for Engineering Geology, if and when it exists. Invitations have already been sent to two persons by IABG).

(c) Possibility of the Quarterly Journal of Engineering Geology becoming the official journal of the IABG.

(This suggestion has been broached informally by them, but any action will have to come from us).

The views of the Council of the Geological Society of London were transmitted to the Secretary General of IABG in August 1970. It was felt that the Committee of the Engineering Group of the Geological Society could and should fulfil the function of a "British National Committee for Engineering Geology". Accordingly, an approach would be made through the Royal Society (the necessary official channel) to have the committee accepted and recognised by IABG as the British National Sub-committee for Engineering Geology, with the Chairman as the official representative.

At the Paris Congress in September 1970 the position was made clear to the IABG Council. I attended as observer, and Mr. Glossop successfully argued the case against the London section of AEG becoming the National Association representing engineering geology in the United Kingdom.

Unfortunately we were unable to present alternative proposals and the matter rested until 1972. It was accepted, however, that the Geological Society representative should be the U.K. delegate to IABG. This would of course be an informal arrangement until the question of official adherence was settled.

September 1970 marked an important event in the development of IABG; the Constitution had been agreed and would come up for ratification at the IUGS meetings at the International Geological Congress in Montreal in 1972. This would affirm the position of IABG in Engineering Geology. It was desirable that the question of U.K. adherence to IABG should be settled before then.

THE NEXT PHASE: 1972 - 1974

In August 1972, Mr. Glossop proposed that I should follow him as a member of the Executive Committee of IABG, as his term of office would expire at the Montreal meeting in September.
Immediately after the Montreal meeting a direct approach was made by the London group of AEG "about liaison of some sort between the Engineering Group and the AEG to form a British IAEG Committee". Coming unexpectedly after two years of disillusion and inactivity, this approach revitalised efforts to find a satisfactory solution to the problem. Discussions were reopened, and opinions were sounded from various individuals and groups as to the best way in which a closer, more formal contact could be arranged between Engineering Geology in the U.K. and IAEG.

Questions were asked in January 1973 as to what was happening in the U.K. with regard to IAEG, and with a deadline of May - the time of the next joint meeting of the Secretariat of ISSMGE, ISRM and IAEG - a direct approach was made to the Royal Society.

On 28th February 1973 I recalled that during the International Geological Congress in Montreal "the question of some form of affiliation of United Kingdom interests in engineering geology to the international association was raised. I would like to see the problem resolved, one way or the other, and set out below some comments and suggestions on how this might be done:

(i) The present position with regard to the London group of AEG, that it cannot be accepted as representing United Kingdom interests, remains unchanged.

(ii) For financial and other reasons it is not sensible to consider the membership of the Engineering Group of the Geological Society as in any way representing a national association which could become affiliated to IAEG.

(iii) I think there is a strong nucleus of possibly 40 to 50 engineering geologists in this country, each of whom would be willing to join the IAEG either as an individual member or as a group member and pay the annual subscription. Care would have to be taken to ensure that this "association" of people fairly and fully represented engineering geological interests in this country. This I feel could be ensured by discussion with the committees of the Engineering Group the British Geotechnical Society and the Association of Engineering Geologists by a direct approach to the members of these various groups. Many engineering geologists are members of all three groups.

(iv) There remains, at least, the question of the organisation of this "association". I have emphasized throughout my informal discussions with members of the executive committee of IAEG that any form of "National Association" representative of engineering geologists in this country would have to be accepted by and organized through the Royal Society and the National Committee for Geology. It seems to me, and I put this forward as a suggestion for your consideration and comment, that the establishment of an 'Engineering Geology Subcommittee' of the National Committee for Geology would best meet the
needs of international cooperation in this branch of geology.

I would welcome comment on the need for some such association as outlined above and on my suggestion on one way in which to bring it about".

This letter eventually produced the desired, but rather delayed, effect: an ad hoc meeting was arranged at the Royal Society to discuss U.K. "affiliation" to the IAEG. On the 31st October 1973 a meeting took place of representatives of the Specialist Groups of the Geological Society of London, the Chairman of the Engineering Group of the same society, representatives of the British Geotechnical Society and the Institution of Civil Engineers, and myself. The meeting had in fact, been requested by the British National Committee for Geology on 28 June 1973.

A satisfactory outcome to the meeting, setting out the potential role of the Royal Society was summarized as follows:

"Since the meeting was addressed to the possibility that the U.K. might be formally affiliated to the IAEG by virtue of a national subscription by the Royal Society, carrying with it the implication that there would be an appropriate national committee (a subcommittee of the British National Committee for Geology), the apparent advantages for this solution, could be summarized as:

(i) that the U.K. would thus be formally entitled to a vote on the IAEG Executive Committee.

(ii) that there would then exist a national focus (a postbox) for discussing matters pertaining to the IAEG - as opposed to scientific expositions which were already well catered for by the B.G.S. and the Geological Society's Engineering Group.

(iii) Simplification of procedures in communications between the IAEG and the U.K.

It was apparent, however, that the adherence of the Royal Society to the IAEG would not necessarily dispel the additional problems that had been outlined, particularly the desired amalgamation of the three sectional interests involved in engineering geology in the U.K. and the problems of the IAEG Secretariat in communicating with its individual members.

Consequently, it was agreed that it would be precipitate to recommend to the Royal Society that an Engineering Geology Subcommittee should be established. Instead, it was agreed to recommend that a preparatory panel be set up made up of representatives of the British Geotechnical Society (2 - one each for ISSMFE and the ISRM), the Institution of Civil Engineers (2 - one for ICOLD and one for IAEG); the Chairman and Secretary of the Geological Society's Engineering Group, one representative of the London Section of the (American) Association of Engineering Geologists, together with the Secretary General of the ISSMFE (Professor K. Nash), and the unofficial national delegate to the IAEG (Professor Dearman). It was further recommended that the preparatory panel be regarded, in the interim, as the official U.K. link to the IAEG and that the Royal Society should subscribe accordingly ($300 per annum) to the IAEG.
Nothing further happened, and I continued my discussions with interested parties on the possibility of an independent solution. Then, quite unexpectedly, there was more official activity.

A further meeting with wider membership was arranged for 25 November 1974. This was to be a preparatory panel to make recommendations to the British National Committee for Geology. Included on the panel were representatives (in a personal capacity) of the:

- Geological Society of London, Specialist Groups Committee (as Chairman)
- The British Geotechnical Society (BGS)
- The Association of Engineering Geologists (AEG)
- Institution of Mining and Metallurgy (IMM)
- The Engineering Group of the Geological Society
- The Hydrogeological Group of the Geological Society

with two other members: Professor J.L. Knill and Professor W.R. Dearman.

As a result of the meeting the recommended solution was that it should be unanimously agreed to recommend to the British National Committee for Geology.

"That a Subcommittee for Engineering Geology be established to correspond to the International Association of Engineering Geology, on the assumption that the collection and payment of individual subscriptions to the IAEg would be undertaken by another body and that there would be no requirement for national dues to be paid by the Royal Society to the IAEg and also agreed that the Engineering Geology Group of the Geological Society be invited to undertake the responsibilities for collection of U.K. subscriptions to the IAEg."

**THE FINAL PHASE: 1975 - 1982**

On January 9, 1975, I received a letter from the Executive Secretary of the Royal Society telling me that approval had been given by the British National Committee for Geology for the setting up of a Subcommittee for Engineering Geology to:

(a) correspond to the IAEg;
(b) nominate U.K. representatives to IAEg plenary meetings;
(c) report to the National Committee for Geology.

I was invited to undertake the Chairmanship of this new Subcommittee, which was independent of all the affiliations that engineering geologists had in the U.K. to various societies, both professional and otherwise.

The initial meeting of the Engineering Geology Subcommittee was held at the Royal Society on 14th October 1975. Affiliation with the IAEg had been accepted by the IAEg Council at its meeting in Krefeld on 11th September 1975.

The Engineering Geology Subcommittee of the British National Committee for Geology

The subcommittee continued to work successfully until May 1980. At that time the subcommittee discussed the options for its future as a response to a review being undertaken by the Royal Society on the role and need for its various subcommittees.

On 30 September 1980, the Chairman of the subcommittee prepared a memorandum setting out the options:
"The Engineering Geology Subcommittee was set up in 1975 to:
(a) correspond to the International Association of Engineering Geology (I.A.E.G.)
(b) nominate U.K. representatives to the I.A.E.G. plenary sessions
(c) report to the National Committee for Geology

The Subcommittee first met on 14 October 1975 at the Royal Society.

This action followed involved discussions, that had gone on since 1968, on the desirability of establishing a U.K. National Group of the I.AEG. Because of conflicting interests, Mr. R. Glossop, then Chairman of the Engineering Group of the Geological Society of London, had approached the Royal Society. Following discussions with the Executive Secretary, an ad hoc meeting on Engineering Geology was held at the Royal Society on 31 October 1973. As a result it was agreed to recommend that a preparatory panel should be established, and an ad hoc meeting was held at the Royal Society on 25 November 1974. Under the Chairmanship of Professor W.S. Pitcher, those present at the meeting effectively represented, in an unofficial capacity, engineering geology interests in the U.K.

At the meeting on 25 November 1974 it was unanimously agreed to recommend to the British National Committee for Geology that a Subcommittee for Engineering Geology be set up. Eight potential members were nominated.

The proposals were accepted and the first meeting of the Subcommittee was held on 14 October 1975. Since then there have been, on average, two meetings a year. With the passage of time, the conflict of interests that was chiefly responsible for the setting up of an independent forum for engineering geology has virtually died out, or at least is less vociferous. It now seems appropriate to attempt a reunification of national and international interests in the subject if some of the remaining problems can be resolved. These are, briefly:

(a) The British Geotechnical Society (BGS) already handles British affiliation with both the International Society of Rock Mechanics (ISRM) and the International Association of Soil Mechanics and Foundation Engineering (ISSMFE).

(b) Engineering geological interests in the U.K. are catered for by the Engineering Group of the Geological Society rather than B.G.S.

Affiliation of a British National Group, however constituted, to I.AEG through the British Geotechnical Society would be a tidy solution, having in mind the links already established with ISRM and ISSMFE. The Engineering Group has a representative on the BGS Committee.

However, the earliest discussions had always been concerned with closer collaboration between the Engineering Group of the Geological Society and the I.AEG and this would have been a generally favoured solution.

It seems appropriate now to reopen the subject of the Engineering Group, through its Committee, becoming the "British National Group" affiliated to the I.AEG.

After sounding opinion, a direct approach was made to the Chairman of the
Engineering Group, Dr. P.G. Fookes, and to
the President of the Geological Society,
Professor Howel Francis. Their response
was sympathetic to the proposal that the
Engineering Group should take on the role
of National Group for purposes of IAE
affiliation. It was recognised that there
would need to be a representative of the
Engineering Group Committee serving on,
or at least attending, the meeting of the
National Committee for Geology in order
to retain engineering geology links in
the U.K. through to the IAE.

For this purpose, the Engineering
Group of the Geological Society of
London becomes the appropriate 'National
Committee' on January 1st, 1983.

AND SO THE SAGA ENDS.

ACKNOWLEDGEMENTS
Quotations have been made freely from
personal files of correspondence over the
years, and these sources are acknowledged
without identifying them.

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A HISTORY OF DEVELOPMENT AND STATE-OF-THE-ART OF ENGINEERING GEOLOGY

HISTOIRE DU DEVELOPPEMENT ET L'ETAT MODERN DE LA GEOLOGIE DE L'INGENIEUR EN URSS

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ABSTRACT

In the middle of the XIX century in Russia the monographs of civil engineers describing properties of soils as foundation for various structures were issued. Eminent geologists were involved in the railway line prospecting and were studying the geological processes from the physical viewpoint. In the beginning of the XX century they introduced a term "geological technical prospecting". In Russia a period till the end of the 20th was a prehistory of engineering geology. Engineering geology, as a science, was shaped in the Soviet Union in the 30th primarily in connection with hydrotechnical construction.

At present three basic developments are distinguished in engineering geology; soil and rock engineering, engineering geodynamics and regional engineering geology. The primary task of engineering geology is to study the geological environment for the purpose of its rational utilization.

RÉSUMÉ

Les premières monographies avec la description des propriétés des sols comme fondement des différentes constructions ont été publiée en Russie au milieu du XIX siècle. Ces recherches, y compris les recherches pour la construction des chemins de fer, ont été réalisées par les plus grands savants étudiant les processus géologiques de point de vue physique. Au début du XX siècle ils ont introduit la notion "recherches géologie techniques". Pour la géologie de l'ingénieur en Russie la période jusqu'à 1920 année a été une période préhistorique. En Union Soviétique la naissance de géologie de l'ingénieur comme une science autonome dans les 30èmes années du dernier siècle a été lié avec les constructions hydrotechniques. Pour l'instant dans la géologie de l'ingénieur on distingue trois directions principales; études des sols; études des processus géodynamiques et problèmes régionales de géologie de l'ingénieur. La tâche principale de la géologie de l'ingénieur - étudier le milieu géologique dans les buts de son utilisation rationelle.

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Certain historical prerequisites are needed for the development of any science. Among these are the social-economic factors manifested as the distinct requirements of a society and accumulation of certain factual material, specific knowledge, the generalization of which, based methodologically correctly, creates science indispensable for mankind.

Each science has its prehistory. The prehistory of engineering geology is subdivided into two stages. Both stages are closely related to construction.

At the first stage the civil and mining engineers were independently studying rocks as foundations medium and material for various structures. It is hardly possible, even approximately, to specify when rocks started to be studied in relation to construction; without this it would have been impossible in ancient times to erect diverse structures, many of which have preserved till nowadays and stir human imagination.

As the onset of scientific research and generalization of accumulated engineering geological material, i.e. the commencement of the first stage of engineering geology prehistory, can be considered the first decades of the 19th century. This was, by far, connected with the industrial capitalism development in Europe, and, in particular, in Russia. The construction of factories, plants, dams and other structures called for the most rational solutions: sufficient reliability at minimum expenditures. This could not have been achieved without rock studies, therefore civil engineers began to give more consideration to them. Special chapters appeared in their monographs, characterizing rocks for construction purposes. The example of such monographs are works published in the first half of the 19th century: D.Lachinov "A reasoning on the arrangement and stabilization of dams" (1816); M.Gersevanov "Lectures on the marine structures" (1861); V.Karlovich "Foundations" (1869), etc. In these works rocks received the name "soils".

For generalizing the accumulated construction experience and using it in complicated conditions, the civil engineers themselves had to elaborate a classification of soils, describe their peculiarities, characterize soil properties, consider the impact of geological processes on various structures.

The second stage of engineering geology prehistory is connected with the involvement of geologists into the construction prospecting (beginning with the 19th century to the 20th of the 20th century). At this time geologists started to be involved into solving problems related to the construction of railways, canals and other large structures. Many prominent scientists were among the geologists consulting construction, namely: W.Smith (England), Ch.Burkeley (USA), I.V.Mushketov, V.A.Obручев, A.P.Pavlov (Russia).

Particularly emphasized should be the importance of works of I.V.Mushketov - Professor of the Petersburg Mining and Railway Institute, the author of the text-book "Physical geology". He took an active part in prospecting the Krug Baikal railway route.

In the beginning of the 20th century in Russia there appeared a number of geological works related to railway prospecting, the names of which used a term "geological-technical prospecting" (Ditmar, 1908; Karakashe, 1908; Rengarten, 1914) or "technical-geological" (Ilov, 1904, 1911). In 1903 the I Congress of the practical geology workers discussed the scientific level and efficiency of the geological technical works on the routes and new construction sites (Shvetsov, 1981). All this testifies to the fact that geotechnics was born in Russian at the beginning of the 20th century, works of Mushketov fostering this.

In "Physical geology" I.V.Mushketov gave much consideration to the exogeneous geological processes, he had awakened interest in them in many Russian geologists. Significant progress in this regard was achieved by the outstanding representative of the Moscow School of geologists A.P.Pavlov. In 1903 he published a work "Landslides in the Simbirsk and Saratov Near-Volga region", where, based on the material accumulated during railway...
prospecting, given is a characteristic of this elaborate geological process.

A.P. Pavlov, likewise, worked out the soil classification based on considering the soil cohesive forces which influence the soil properties - compressibility and wash-out capacity. This classification was published in 1925 in the book by A.N. Bernatsky "Earth mass stability".

Hence during the geological railway prospecting there had developed a branch, which now can be called "engineering geodynamics", for the geological processes were studied in relation to engineering structures. It can be stated that in Russia as early as prior to the First World War, there were considered problems later published in books by I. Stini "Technical geology" (1922) and H. Ries, T. Watson "Engineering geology" (1925), K. Reddick, K. Terzaghi, F. Kampe "Engineering geology" (1929).

In 1925 the basic work by K. Terzaghi on soil mechanics, which originated at the junction of the physical mathematical, civil engineering and geological sciences, was published. Soil Mechanics considered the general regularities which stem from applying the theoretical and construction mechanics laws to rocks. Therewith, the mechanical soil properties governed by the laws of mechanics and kept within districts calculation schemes, were given priority, while the geological soil peculiarities formed as a result of their genesis, were given less consideration.

In the Soviet Union in 1923 a different trend in studying soils and rocks related to road construction had developed. It acquired the name "Soil and Rock Engineering". The theoretical base of soil engineering was formed by the genetic approach worked out by V.V. Dokuchaev in Soil Science and the works by P.A. Zemyatchensky on clay studies - he had formulated the statement that clay should be studied as a physical body developed in the specific natural historical conditions.

The setting up of the Road Research Bureau in Leningrad in 1923 should be considered the beginning of shaping of soil and rock engineering. The Bureau was headed by N.I. Prokhorov, P.A. Zemyatchensky, N.N. Ivanov and dealt with the study of soils and sedimentary (primarily recent) rocks for road construction. Thus the road soil and rock engineering had originated which later, when genetic approach was applied to studying rocks for other kinds of engineering structures, lost the attribute "road" and obtained a wider meaning - "soil and rock engineering". In 1930 the Chair of Soil and Rock Engineering was set up at Leningrad University and in 1938 - a similar Chair at Moscow University.

By the soil and rock engineering there began to be implied the science studying any rocks and soils as an object of the engineering construction activity of Man, there properties being determined by genesis and post genetic processes and which represent the multicomponent changing in time systems.

This was a new approach to rock studies related to construction giving priority to the geological peculiarities of rock (its composition, structure, texture), and the soil mechanics schemes were considered as a method enabling to explain rock properties needed for construction. Soil and rock engineering began to be developed in the Soviet Union as a natural historical science. This direction was fostered by the works by N.M. Filatov, V.V. Okhotin, V.A. Priklonsky, B.M. Gumenisky, I.V. Popov, S.S. Morozov, et al.

It is well-known that in solving problems related to construction it is insufficient to know only rock peculiarities studied by soil engineering and soil mechanics. Prior to construction, at the stage of site selection and objective assessment of the competing variants, it is indispensable to have a wide range of data on the geological structure of a territory, geological processes, occurring in it and feasible to be induced by construction, hydrogeological conditions, etc. Engineering geology is tackling these problems.

In the Soviet Union engineering geology was shaped as a science at the end of the 20th - beginning of the 30th of this century. At the first stage of its development
of primary importance was the hydrotechnical construction, a part of the Lenin's Plan of Electrification of the country. Its initiation of largely fostered by the works by F.P. Savarensky, G.N. Kamensky, N.F. Pogrebov, I.V. Popov, N.N. Moslov, M.P. Semenov, V.A. Friklonos et al. who took part in the hydroelectric stations prospecting on the Volga, the Dniepr, on the canal track Volga-Moscow, etc. A great contribution into the formation of engineering geology as a science was made by the eminent Soviet geologists E.V. Milanovsky, G.P. Mirchik, N.S. Shatsky et al. In 1929 a Chair of engineering geology was set up at the Leningrad Mining Institute and in 1931 - at the Moscow Geological Prospecting Institute. In 1937 the books "Engineering geology" by F.P. Savarensky and "A method of the engineering geological studies for hydro-technical construction" which had finally secured the notion of engineering geology as a new branch of geological science came out.

Particularly emphasized in the development of engineering geology should be the role of F.P. Savarensky who is justly recognized the founder of engineering geology in the Soviet Union and due to his works in engineering geology and hydrogeology was elected Member of the USSR Academy of Sciences. A great role in developing soil and rock engineering belongs to M.M. Filatov.

At the outset of development engineering geology and soil engineering were comparatively slightly interconnected, but the general regularities of contemporary scientific development, synthesis and differentiation, have changed the course of further development of soil engineering and engineering geology; there has occurred a close interpenetration of these sciences and isolation of regional branches, on the basis of which regional engineering geology was developed. In the USSR by the engineering geology began to imply not a separate scientific discipline, but a new branch in geological science consisting of the three basic divisions: soil and rock engineering, engineering geodynamics and regional engineering geology.

Given such an understanding of engineering geology, I.V. Popov (1959) characterized it as a branch of geology studying the Earth crust dynamics (primarily its upper horizons) related to the engineering activity of Man. The prediction of interaction of geological environment and an engineering structure during its construction and operation was considered as a primary task.

Since 1959 20 years, saturated with diverse historical events, have passed. The rate of the scientific technological progress was growing during those years. In this view, the environmental problem has gained world-wide importance, became a global problem. Particular attention is given to it in the socialist countries.

The importance of engineering geology has grown; it began dealing not only with the construction problems but also with the problems stemming from the necessity to protect and rationally use the environment. In 1966 the Scientific Council on engineering geology and soil engineering was set up in the USSR Academy of Sciences; in 1979, it was reorganized into the Scientific Council on engineering geology and hydrogeology. The USSR National Committee of geologists has an Engineering Geology Division uniting the leading scientists and specialists working in various institutions of this country. In 1968 at the XXII International Geological Congress the Soviet National Group of engineering geology has become an IAG member and since that time is taking an active part in its work.

Beginning with January 1, 1979 the academic journal "Engineering Geology" started to be issued. Its first number was opened by the article called by the common consent of all the Editorial Board members: "Engineering geology - the science of geological environment". This name reflects a modern view on engineering geology of the majority of scientists working in this field.

Nowadays every engineering undertaking should be estimated not only from the viewpoint of feasibility of its implementation, but, likewise, with the view of its
impact on the environment; the construction feasibility and profitability are determined not only by the strength and deformation properties of soil, but also by how one or another engineering structure will be "inscribed" into the natural environment. The efficiency of any engineering undertaking, its economy should be considered keeping in mind changes in the entire environment. The very notion of environment is connected with Man and his activity. The environment surrounding Man is studied by many sciences. A part of lithosphere subject to the impact of Man and determining the character of this impact is now studied by engineering geologists as the geological environment.

By the geological environment we imply any rocks and soils making up the upper portion of lithosphere and considered as the multicomponent system undergoing the impact of the engineering economic activity of Man, this bringing about changes in the natural geological processes and initiation of the new anthropogeneous processes, this, in turn, causing changes in the engineering geological conditions of a certain territory.

Given such a definition of the geological environment, each of the three basic branches of engineering geology obtains a distinct aspect in tackling the problems it faces.

Soil and rock engineering can be determined as the science studying any rocks and soils as the multicomponent dynamic systems changing due to the engineering activity of Man.

The successful development of soil engineering in many respects will depend on an apt combination of the field and laboratory investigations capable to characterize the strength and deformation properties of rocks based on the "microworld" inherent in them.

The basic state-of-the-art problem of soil engineering is a study of rocks as soils from the microlevel to the massif on the basis of a notion of their being the multicomponent systems making up the geological environment.

It should be remembered that without the engineering geological studies of rocks it is impossible to solve successfully problems facing engineering geodynamics and regional engineering geology, for rocks are a major factor determining the development of geological processes both the natural and those caused by the Man's activity, and they are one of the primary engineering geological conditions in assessing a territory.


Engineering geodynamics is studying all the modern geological processes indispensable in estimating particular regions for the purpose of their national economic development, in design and construction of large engineering structures, and likewise the ancient geological processes which influenced the geological structure of a territory.

Therefore the major problem of engineering geodynamics is to predict quantitatively the exogenous and endogeneous geological and engineering geological processes in space, time and by intensity for the purpose of preventing the development or decreasing the hazardous impact of these processes.

The occurrence of the exogeneous geological processes in space is now predicted with high reliability, things are worse regarding prediction by their intensity and even more worse concerning a process occurrence in time. Meanwhile, the rational use of geological environment necessitates to predict geological processes in space, time and by their intensity. This is a very difficult and elaborate task, for it has to be solved with due account taken of the highly diverse factors. But it must be solved and, first of all, regarding the processes of disastrous character - this is demanded by the need to protect and rationally use the environment.

The leading scientists developing this direction are: N.N.Maslov, P.P.Shvetsov, L.D.Bely, I.M.Buachidze, G.S.Zolotarev, F.V.Kotlov, G.A.Mavlyanov, I.A.Pechorkin, V.P.Solonenko, G.I.Ter-Stepanyan et al.
No matter how important is the prediction and study of the geological and engineering geological processes and phenomena, yet more important for the diverse activity of Man is the estimate of the engineering geological conditions of separate regions. Without exaggeration this objective is of the national economic importance, for the modern activity of Man is capable to change not only the state of geological environment but of the entire natural environment over vast territories. The rapid development of the regional engineering geology in this country over the last decade is related to this, its major problem being the cognition of the regularities of development of the engineering geological conditions of large geological regions and prediction of their changes under the impact of the Man's activity.

For the first time in this country the 8-volume monograph "Engineering geology of the USSR" has come out. It gives consideration to the history of development of the engineering geological conditions of all the large geological regions of the Soviet Union, likewise, performed was the engineering geological zoning of regions in accordance with which given was the engineering geological characteristic of the distinguished units; account was made of the construction experience and the natural conditions' changes under the impact of the Man's activity. This capital work was made possible by the works of I.V.Popov, N.I.Nikolaev, M.V.Churinov, G.A.Golodkovskaya, I.S.Komarov, I.M.Buachidze, V.T.Trofimov and many others, which had elaborated the basic principles of the regional engineering geology.

Now there already arises a necessity to compile maps giving a possibility to estimate (and in future, apparently, they will allow to predict) changes in the geological environment under the impact of variable activity of Man, in particular under the impact of the mining industry, land reclamation and hydrotechnical construction, city building and road construction.

The development of engineering geology as a science is in progress. At present the engineering geology of cities, engineering geology of mineral deposits and marine engineering geology are somewhat isolating in the engineering geology. In future the origination of the engineering geology of planets is quite likely to occur. All these trends are developing in this country.

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ENGINEERING GEOLOGY IN THE ECONOMY OF DEVELOPING COUNTRIES

GÉOLOGIE DE L'ART DE L'INGENIEUR A L'ECONOMIE DES PAYS DEVELOPPANTS

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ABSTRACT

Not a single science can do without considerations of its own usefulness. In particular examining its future development possibilities.

The author in this paper on the above-mentioned basis scrutinizes economic role of engineering geology in the developing countries. The papers value originated first of all from its multidisciplinarity.

Main topics: - the role of engineering geology and its connection with civil engineering construction activities in general
- the role of engineering geology in preparations for decision making; a hierarchy of the different variants of investment projects
- consideration of the special conditions of developing countries and economic base of their being interested in applying engineering geology.

The author pays special attention to the paper's fitting in the themes' collection of the congress.

ABSTRAIT

Aucune science ne peut pas s'en passer de l'examen de sa propre utilité, surtout si elle cherche la possibilité de son futur développement.

L'essai analyse de ce point de vue le rôle de la géologie de l'ingénieur dans les pays en développement. Sa valeur provient d'abord de son interdisciplinéité.

Son thématique principal est le suivant:
- le rôle de la géologie de l'ingénieur et sa relation avec l'activité des ingénieurs civils
- le rôle de la géologie de l'ingénieur dans la préparation des décisions
- la base des intérêts des pays en développement dans l'application de la géologie de l'ingénieur.

L'auteur tient compte que l'essai s'ajuste dans l'un des thématiques annoncés du Congrès.

The role of engineering geology and its connection with civil engineering construction activities in general

Engineering geology is a socially important, strikingly useful activity producing tangible economic results, being an up-to-date, interdisciplinary field of science.

Based on success hitherto achieved and further improving its versatility, it is the source of countless possibilities for renewal. Should the usefulness of its activities and functioning be put on the balance, so it is clear that the scientific performances involved are so big that they often have to be and can be evaluated on the level of macro-economy. On account of a complex engineering geological research helping medium- to long-term planning and of its important decision-making role, the afore-mentioned circumstance is most obvious. This is what makes engine-
ering geology particularly important for the developing countries, an efficient means for the development of their economic structure as it is. In fact, it provides an answer to a number of questions of importance for the development programs of these countries, questions that in many cases include both the questions of "how, which way" and "what" or "which choice is more advantageous".

In this function of its, engineering geology, say ventures to stroll, in its peculiar geotechnical "disguise", over to the field of scientific economy. In doing so, it gives an answer to questions not studied yet all-embracingly by economy in the strict sense. In this answer-giving, the proper methods of observation or measurement may happen not to be available in its own arsenal of techniques. Of course, the same shortcomings hold true, from the other side, for the functioning of engineering geology as well. It would often be difficult for us to express our estimates in quantity or in terms of values.

There is already a need for engineering geological research of economic outlook, primarily as far as the complex engineering geological activities are concerned. The evolution of engineering geology is, no doubt, an eloquent example of how science becomes a force of production. On this basis one should seek to be able to predict the size of the social product stemming from this force. The task is still far from being solved. The present paper goes only to the level of problem statement, hinting only at one of the possible approaches. There may be also such an opinion that this question belongs to the problematic of economics geology and that engineering geology is not obliged to scrutinize the utility of its activities. I think, this opinion does not correspond to the rational attitude of these days.

Applied geology and engineering geology have good reason to make an "engineering geological economic geology", i.e. to cooperate with economy. After all, why should it not do so?

Engineering geology is that part of applied geology most closely linked with engineering. Its purpose is to enhance civil engineering building activities, conducted as they are in a geological environment, and to seek build up a most suitable interaction between engineering structures and geological natural environment.

Engineering geological research is a synthesizing activity, one having an attitude towards carrying out complex, interdisciplinary geological and geodynamic studies and tests.

One would then be right to ask, why should the developing countries so badly need to enjoy the benefit stemming from engineering geology? No satisfactory evidence to rely on for a proper answer could be gained, if one started with analyzing the sphere of engineering geological activities, should it be its field of action taken in the broadest possible sense, in order to approve that all this cannot be dispensed with.

When asked, what we usually do, we list, as a rule, the items civil engineering requires from applied geology so as to render its performance more efficient. This can be done in the case, when in a given place some definite civil engineering building activity is to be carried out, i.e. decision has been taken to erect a structure there. The general practice virtually shows that in such cases the engineering geological studies immediately precede the building, or are run synchronously with it and that they provide building engineers with direct data files on the results of the pertinent soil - and rock mechanical and geotechnical tests. Of course, this is a kind of explanation in itself, but we are able to say even more than that. What is it the case, when engineering geological research is progressed not only by one /or maybe half/ step, but possibly "by two steps", ahead of the progress of civil engineering activities? In this case, in promising, development areas -- in which the civil engineering building activities are expected to be started within the span of time of medium-term planning -- a complex engineering geological research is previously performed. Such projects are launched, in the socialist countries, e.g. in high priority development areas of medium-term /five-year/ plans, the municipal areas of cities, bustling recreational districts, major engineering structures and industrial plants /e.g. highways, airports, quarries, surface mine pits, drops, etc./ and their neighborhood. In such cases the efficiency must be evaluated in a complex way, interdisciplinarily, on the level of national economy.

In general, the more disrupted in time an engineering geological activity from a concrete building project is, the better the information resulting from engineering geological research can be used during the planning of engineering building works and operations and while taking decisions concerning the location of structures or housing or industrial objects, and the less can it be useful for the virtual execution of the project plan. This is the very kind of usefulness which already leads us to a never approach to the usefulness of engineering geology. In case of countries comparatively deficient of funds and usually facing difficulties in financing in general, to suggest the use of engineering geology may seem an encouragement to a lavish spending of funds in the eyes of an observer not familiar whit
the subject, whereas it is just the contrary that is the case. Our countries are not so rich to deprive themselves of the services of a purposefully controlled and efficiently applied engineering geology. As ascertained in quite a number of cases, an investment project, when launched without proper preparative measures and economic feasibility comparison and evaluation based on preliminary studies and tests by skilled experts, will "pay" a waste of money. Not to speak of those cases when the lack of preventive measures may threaten the mere existence or the safety of operation of a civil engineering object, the meager resources are exposed to the risk of being endangered or lightheartedly dissipated even in the so-called "medium-risk" cases. And the funds thus wasted might have been used for financing hospitals, schools or social amenity projects elsewhere.

The aim of engineering activities is to satisfy particular preferred needs of society. To be able to formulate real aims /through the economic management organizations of a particular country/, society must create a harmony between development needs and development possibilities /Fig. 1/.

DEVELOPMENT NEEDS AND POSSIBILITIES /technical and geologic or economic/ FOR ENGINEERING PROJECTS TO BE ERECTED IN THE GEOLOGICAL AND NATURAL ENVIRONMENT

The engineering and geological variables in Fig.1 are shown plotted along the same coordinate axis, forming one and the same strip of the "plane of possibilities". The same is the case in reality, as engineering construction and engineering geology are prerequisites for each other, both parties acting to achieve identical goals. Engineering geology makes efforts towards obtaining a harmony of the geological-natural environment on the one hand and the human product, the engineering object, on the other. If the natural environment versus engineering object relation is approached in an up-to-date manner, so engineering geology, determined to ensure the longest possible conservation of the planned object and its safe functioning as it is, should be at least so much determined and endeavoured /and is so/ to ensure, through most strenuous efforts, the same advantages for the natural environment concerned as well and to conserve them in the long run.

Consequently it stands to reason that the raison d’être and aim of engineering geological research, data processing and evaluation enhancing the resultativeness of the engineering activities can be explained and formulated in the context of a rentability analysis of the engineering construction activities. No doubt, in its various fields of action, engineering geology will affect the rentability of the engineering activities. This positive effect is a set of extremely heterogeneous elements. Clearly enough, the extent of heterogeneity in this respect can be best compared to the heterogeneity of the field of action of engineering geology. Restricting ourselves to the most striking and obvious effects and influences, we can list the following: engineering geology enhances the safety of operation of construction projects and objects; by prolonging the time-span of functioning of these plants it directly reduces the economic risk of the investments, makes the planning reliable, prepares and substantiates decisions on the matter of investment, achieves a considerable reduction in investment cost, establishes a hierarchy of location site alternatives, etc., etc., etc. A typology of some major groups of engineering geological activities according to their economic character is presented in Table 1.

Economic character of engineering geological activity groups

<table>
<thead>
<tr>
<th>Type of engineering geological activity</th>
<th>Economic character, aim of the activities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Choice of location, hierarchy-evaluation</td>
<td>To draw a profit from the activities/to maximize the differential rent, to reduce the risk/</td>
</tr>
<tr>
<td>To answer the &quot;question of &quot;how&quot;; Social usefulness</td>
<td></td>
</tr>
</tbody>
</table>

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to assure the harmony of the geological natural environment and the planned technical object or structure.

Analyses, tests, information services
To provide services /service function/

There are two or more possibilities for characterizing the relationship between civil engineering activities and engineering geological research. One can differentiate according to the role of engineering geology in connection with the technical object or structure involved. In this respect, an engineering geological research may be connected with decision-making as to civil engineering projects to be launched. It may be associated with drafting and designing stage of a project or the stage of its execution, or possibly even with the reconstruction of a technical object. In general there exists an IF-THEN relation between engineering construction activities and engineering geology. Whenever a particular engineering problem is to be solved, engineering geological activities of that particular type must be carried out. The relationship is not represented by a matrix with mere units in its main diagonal and with zero in the remaining \( x_{ij} \) places, but, instead, if \( i \) is the line-index and \( j \) the column-index and if \( j=1, ..., n \) represents the engineering construction tasks and \( i=1, ..., m \) the engineering geological research methods, then a column sum, \( \sum_{i=1}^{m} x_{ij} \), representing the assemblage of engineering geological methods to be selected for the given \( j \)th construction /building/ problem will pertain to each \( j \) position. In other words, the types of engineering geological activities in this case must be replaced by research methods.

Although the type of activity mentioned in the Congress circular does not fully correspond to this purpose, I have attempted at finding a relationship between this classification, well-known as it is to all of us, and the economic effects expressing the usefulness of the engineering geological activity /Table 2/.

The density of symbols in the tabulation expresses the intensity of the economic effects involved. The maximal values of symbol density place a marked stress on some types of engineering geological activity /three of which have been framed/, types characterized by an extremely great influence on the economic result of the engineering construction /building/ activities concerned. It is worthy of attention to point out how different the first main group of activities from the rest is, a difference that can be clearly correlated with the peculiar aim of this kind of task or project, with the different nature of the task involved and the complexity of the approach to it. I should also like to call attention to the "different optics" with which the table can be read when the two table headings are interchanged. Notably, inasmuch as we intend to ensure that any of the economic effects should be felt from all aspects, we have only to trace the corresponding column over its total length and the activities needed to achieve the goal "will be selected". If we want to be very scientific, we may even say that Table 2 describes a sophisticated stochastic functional relationship whose result variable is the social usefulness of the engineering geological activity and whose two explanation variables are the type of the engineering geological research activity involved and the kind or quality of the economic effect of engineering geology. All this is quite logical, for the economic effects considered are unequal in terms of intensity, being arranged in a kind of hierarchy. This hierarchy, however, has not been established for the activities, lest the compilation should change. In spite of this, to differentiate as to the relative value of the particular activities may be approved. Nevertheless, let us prefer not to make a distinction of this kind now, and to add, on top of that, to the question that to have the two different scales is not the most difficult problem in studies of this kind.

Hereinafter let us discuss, out of the economic effects listed in the headings of Table 2, two functions of engineering geology I judge to be most important of all: the role it plays in decision-making on the matter of investment projects on the one hand and the role in maximizing the differential rent on the other. Before doing so, however, a few words must be said about the information services provided by engineering geological research.

The activity types of Table 2 represent, each, a particular source of information. What is the rate of flow of information, the efficiency of obtaining and propagating them? Both are limited. Despite the impossibility of imposing strict normatives on every type of engineering geological activity, of specifying the length of time, the means, the energy, etc. to be involved, a scientist is always aware of his having to do with a case in which the input is not proportional with the
output. And we always stop when reaching to a particular point.

Economic effects of various types of engineering geological activities

<table>
<thead>
<tr>
<th>Main groups of engineering geological activities</th>
<th>ECONOMIC EFFECTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Types, activity fields and branches of engineering geological research</td>
<td>Maximizing the differential rent</td>
</tr>
<tr>
<td>Intensity of effects: high x medium . low sized</td>
<td>Increase of output /proportion of operation time span</td>
</tr>
<tr>
<td></td>
<td>Reduction of investment cost</td>
</tr>
<tr>
<td></td>
<td>Reduction of operation cost</td>
</tr>
<tr>
<td></td>
<td>Increase of production safety</td>
</tr>
<tr>
<td></td>
<td>Increase of production risk</td>
</tr>
<tr>
<td></td>
<td>Breakdown of reliability of planning projects</td>
</tr>
<tr>
<td></td>
<td>Intensification of the realization of the natural environment resources</td>
</tr>
<tr>
<td></td>
<td>Contribution to the protection of the natural environment</td>
</tr>
</tbody>
</table>

1. Engineering geological studies for environmental evaluation and development:
   1.1. Rural areas
   1.2. Urban areas
   1.3. River Valley project areas
   1.4. Sea Coastal areas
   1.5. Mountainous areas
   1.6. Desert areas

2. Engineering geological problems of tunnelling and excavation of cavities:
   2.1. Prediction of hazards and problems of tunnelling and excavation of cavities.
   2.2. Quantitative appraisal of hazards and problems of tunnelling and excavation of cavities.
   2.3. Natural stress fields and excavation of tunnels and cavities.
   2.4. Rock loads and supports for tunnels and cavities.
   2.5. Classification criteria for tunnelling/excavation media.
   2.6. Environmental impact of construction of tunnels and cavities.

3. Soil and rock as construction material:
   3.1. Selection criteria of material for earth and rockfill dams, road and railway embankments, ornamental building stones.
   3.2. Testing of materials for use as 3.1.
   3.3. Performance of materials in structures as listed in 3.1.

4. Engineering geological problems of natural and man-made lakes:
   4.1. Assessment of factors contributing to siltation of natural and man-made lakes.
   4.2. Control of factors contributing to siltation of natural and man-made lakes.
### 4.3. Shore processes relating to natural and man-made lakes.

### 4.4. Assessment of stability of slopes encompassing natural and man-made lakes.

### 4.5. Control and remedial measures for ensuring stability of slopes.

### 4.6. Reservoir induced seismicity.

### 5. Engineering geological problems of sea-coast and shelf areas:

#### 5.1. Assessment of factors controlling coastal and shelf erosion.

#### 5.2. Coastal and shelf erosion control measures.

#### 5.3. Harbour installations and their protection.

#### 5.4. Off-shore developments—their investigation and assessment of related problems.

However, not only the engineering geologist is compelled to stop! During planning, when the cost to be expected for a project is dealt with, a similar problem is faced. Beyond a definite point, the input is not in proportion with the output and the reusability of our price forecast will not too much increase even if all the methods for price analysis and forecast are verified forcibly through a maze of price statistics. Consequently, the circumstances will then stiffen, by and large, according to Fig. 2 and we have to take decisions as to the question of "how" for the construction of single projects, i.e. as to the type of structure or technical object to be built in a given place, the method, the technology and means by which the harmony between the object and its geological environment is ensured, etc.

Problems due to the lack of engineering geological information in selecting a building site

The engineering geological basic information needed for comparing the potential location variants of an engineering construction project /determining the pertinent comparable indices/ is, a priori, not available. Moreover, considerable difficulties may be encountered even in the case, if the best intended efforts are made in order to carry out the engineering geological research. The question to be answered while evaluating jointly the individual variants is usually not concerned with the method by which their economic feasibility or unfeasibility are checked, but usually with the conclusion that we are able to deduce from the results of engineering geological research. Accordingly, the problems due to the partial lack of information are natural concomitants of the comparisons of this kind.

Listed in the order of decrease, the following three degrees of information deficiency can be faced with:

1. No engineering geological research is carried out at all.
2. The engineering geological research needed is performed only partly or with a delay.
3. The necessary engineering geological work is carried out, but to acquire the missing information by further research would be nonremotely.

In all three cases, the deficiency of information is only partly responsible for the limitations, but it does not make impossible to compare the different variants. Naturally, the reliability, or risk, grow exponentially with a 3-to-1 trend.
Some of the lacking information are due to such uncertainties, obscurities that concern, equally or almost equally, all the concurrent variants involved. Let us quote for example, the lack of information on the possibility of occurrence of natural disasters. An earthquake may afflict areas of the size of a nation, while the relevant forecasts are rather uncertain. Such kinds of possibilities, however, will equally concern all of the variants, in other words, they do not too much reduce the chances of the correct determination of their relative rentability.

Much more concern derives from information gaps unequally affecting the different variants. To classify them, however, would be a difficult task. Namely, if the above three possibilities are examined from the viewpoint of the geotechnical characteristics of the individual sites, usually different as they are, quite a number of combinations is conceivable and the comparison will become extremely difficult.

The situation can be improved, if the possibilities listed under paragraphs 1 and 2 are categorically excluded and if we shall do our best according to paragraph 3. The problems to be confronted in this connection, given the dissimilarity of the fundamental circumstances, the different technical and technological schemes implied and the need for assessing the complex of these factors in terms of economic feasibility, will be quite enough even in this case. Consequently, it is advisable to carry out honestly the engineering geological studies in each particular case, as any information, should it be pregnant with more or less uncertainty, is even in the worst case a better choice than the total lack of evidence. To scrutinize the degree of uncertainty, however, is extremely important in these cases.

A rather grave situation is the case when an essential, relevant parameter of a given variant is unknown. This situation may hold true even for case 3, since there may occur circumstances under which very expensive field tests and studies would be needed and their cost would render the investigation nonrentable. Two alternatives are suggested for these cases:

a/ A kind of estimate is carried out as to the potential lower and upper limits of the unknown parameter/s and the comparison is carried out accordingly.

b/ What we determine is the unknown parameter should have so as to enable the uncertain variant to become competitive with the concurrent variants, and it is after this that we examine the probability of existence of this value.

Because of the lack of information we have to be contented with an evaluation by qualitative characteristics /e.g. with statements like "substantially more favourable", "somewhat better", "roughly equal", etc./. Such statements, however, are suitable for decision-making purposes.

Worse is the case, when we can formulate our judgement only in a conditional mood /e.g. in this range of parameter values the variant in question might be considered to be of roughly equal rentability with respect to one or another competitive variant/.

If the basic information available is unreliable to the extent that even an approximate image cannot be obtained, so it may be suspected that the research has been carried out inefficiently or that we may have to do with case 1.

The role of engineering geology in preparations for decision a hierarchy of the different variants of investment projects

All in all, the project/location/ variants reducing the chances of one another during comparison are competitive to one another and form the category of a "relevant market" for engineering geology. It is very essential to see clearly that this assemblage of variants is not a priori available and that the investment project variant chosen and to be implemented cannot come away with the palm independently from an engineering geological judgment of the matter. Engineering geology may play a quite considerable /though varying following the cases/ role in establishing the hierarchy of variants; in other words, engineering geology is an active partner on this relevant market, a partner that manipulates the market, or at least it must do so, in order to get things done.

The hierarchy of variants too should be based on relevant variables, so-called key parameters. Cost estimates can be enhanced, on the part of engineering geological research, by answering two fundamental questions.

1. How much will be the least favourable value of each key parameter at the individual location variants? To know this figure is necessary for estimating the returns deducible from the differential rents pertaining to the individual variants.

2. How much may be the error of determin-
nation of the least favourable key parameter? To know this figure is needed for determination of the size of risk.

One must not be misled, of course, to believe that the optimal site of an engineering project selected from construction and operation considerations will, of necessity, coincide with the optimum based on engineering geological considerations. This may be, in fact, only in special circumstances the case and the engineering geological parameters are in midfield among the parameters basically controlling the cost of erecting an engineering structure and its returns. Exceptions to the rule are the cases when some parameters are crucial and when the very existence, the maintenance and the safety of operation of the whole object or plant /e.g. large dams, high-rise buildings, etc/ depend on them.

Consequently, with their clearing the geotechnic fundamentals the engineering geological investigations are able to establish a kind of rentability hierarchy between the various alternatives that may occasionally come in question. Naturally, this hierarchy is based on engineering geological principles, for it reflects the size of the differential rent in terms of engineering geological standards, a rent computed with reference to the most unfavourable of all alternatives that may come in question and that are judged indispensable even from other, non-engineering geological, considerations.

In terms of value, the difference between the most and least favourable alternatives of rentability by engineering standards is not always the greatest of all the various factors influencing rentability. For example, the presence of favourable or unfavourable engineering geological fundamentals may happen to affect only 20% worth of change in investment cost, whilst differences, say, between the different /more up-to-date or more backward/ execution technologies may provoke an increase or reduction by 40 to 50% of the planned cost of investment. It is therefore essential to interpret the relationship between the cost-influencing factors and the cost /or output or profit, respectively/ as a multivariable stochastic relation, where the engineering geological background accounts for only a smaller fraction of the independent variables and where even some multicollinearity is supposed to exist between the independent variables. The cost-influencing role of engineering geology, the size of its influence may vary from one object to the other and the same holds true for the size of multicollinearit

ty, a factor difficult to eliminate.

All in all, the above discussion has shown convincingly that the knowledge of the engineering geological fundamentals is crucial, for the engineering geological characteristics directly affect the rentability of the engineering projects and, in case of major projects, even a few per cent of the investment cost amount to a sizeable sum of money.

**Consideration of the special conditions of the developing countries and economic base of their being interested in applying engineering geology**

In developing countries engineering geology can achieve its goal of being properly involved in the economic life of the country only with specific features dependent on the productive forces and the production conditions of these countries. That the existing economic structure, its level of development and the possibilities to improve them are strict regional limitations to any economic activity seeking to rely on domestic resources, cannot be left out of consideration. That the gradual elimination of the dependence of developing countries on foreign resources in the present period of universal slump will be increasingly more difficult to achieve, in order to close up to the economically advanced regions, should certainly be reckoned with. Everybody must be aware of the fact that economic development planning has to calculate with factors like the conditions of production and the present state of the social superstructure as a reality that can be assessed in strict terms. Unless these factors are taken into consideration, economic planning will be degraded to the level of an Utopian approach.

Weighing all these determinants together, the fact is that the field of action for engineering geology in the developing countries is rather restricted, its possibilities are limited in spite of a theoretically high demand for its contributions, and they largely depend on our ability to exploit the high-grade adaptability of engineering geology to the new conditions met with. If we are convinced of engineering geology’s ability to serve to the benefit of our countries, we have several tasks to solve:

1. To survey the existing needs on a large scale, or to encourage people to express their interest, to demonstrate the economic utility of engineering geology by assuring proper publicity.

2. To ensure the needed number of engineering geologist from both domestic and foreign sources.
3. To seek or develop such fields of activities that may enable the engineering geologists to draw fair profits by a maximal use of the local endowments and at the cost of modest financial means.

The peculiar two-faced nature of the economic structure of the developing countries frequently produces largely different conditions for engineering geological activities. Notably, the tasks, the economic role and the profits of engineering geology may largely differ in dependence on whether it has to do with big investment projects launched by the State or by national companies of solid capital or financed with foreign funds, or it is involved in major centralized development programs or in projects financed from local funds, being launched by local administration agencies, minor companies, cooperatives or residents' associations, etc.

In case of major State-financed programs /provided that they have been carefully prepared/ the proper timing of the engineering geological works and the engineering activities can usually be achieved. More difficult and for the most part unanswered is the question of how engineering geology can help the peoples of developing countries when its duties consist of assisting and orienting engineering project relying on local resources and aimed at exploiting local potentialities. Local resources are to be understood in this case as the funds and material assets of minor or medium-size companies, the economic management organizations or agencies of the local administration and the cooperatives and residents' productive groups or economic associations, i.e. the investment of minor funds to erect engineering structures is meant. To exploit local potentialities is the most easily viable alternative for the economic upswing and progress of the developing countries. This principle is in harmony with all the circumstances that usually control the success or failure of regional development plans. In addition to the natural resources, these circumstances include the educational level of the population, its ability to introduce a technology that is by just a degree more advanced than that already existing /it is no accident that I do not use the term up-to-date or, the less so, most up-to-date!/ , a fundamental and basic determinant of a gainful exploitation of the planned objects.

Let me quote in this connection by experiences in Central Asia: on the steppes of Mongolia, to ease the problems of water shortage for a livestock-holding population, projects have been launched with the aim of developing a system of so-called whim-wells. These, as a rule, ensure local people an access to the water hidden in the debris of filled-up streams or creeks. In Outer Mongolia /People's Republic of Mongolia/ I met several times well-digging teams consisting of a few members, who were locating their wells quite systematically and with an imposing workmanship. This type of well is a very simple one and the herdsmen can operate them without any difficulty. Nevertheless, regrettably enough, a considerable part of the wells, still comparatively new, are useless owing to failures that might be easily eliminated and repaired. The problem is that the well-digging team is working somewhere far away from the well in question and, in lack of proper instruments, the residents are unable to repair the wells regularly. Under such circumstances, because of the simplicity of their maintenance, the dug-wells are still competitive to the whim-wells in spite of the numerous advantages of the latter.

Under such and similar circumstances an engineering geologist must be widely familiar with the local potentialities and realities so as to be able to judge the very kind of contribution his discipline can make to achieve a proper hierarchy of engineering projects to be launched /I mean this in the broadest possible sense/. To judge the importance of purely professional, technical aspects is an important and difficult task requiring an unbiased approach to scrutinize all the bearings of meeting local demand including even the relations of people involved in the production process. This should be the base upon which the engineering geologists will decide which investment alternative is more and which less favourable. Since the special requirements of engineering geology usually remain in the second or third ranks in being enforced, to search for alternatives directly ensuring the greatest advantage, i.e. alternatives in which the economic advantage stemming from the presence of favourable natural characteristics comes close to the optimum, is of particular significance. This is all the more logical, as in case of such projects of local significance the difference between the input cost of the least favourable alternative and the most favourable one is usually much smaller than it would be the case with State or big company projects. In fact, much of the difference between the alternative variants is due to the dissimilarity of the physical characteristics of the sites, for a choice between differentiated technologies, transporting materials from greater dis-
tances or employing a more qualified and, consequently, more expensive labour force /e.g. from abroad/, cannot even come in question. Thus the physical or natural conditions are strikingly predominant as compared to the remaining cost-influencing factors, so that the differential rent controlled by them may be crucial in the judgement as to rentability. Should this too not be particularly significant, so we still can resort to using the conventionally efficient method of a good merchant, i.e. to increase the business turnover, in other words, to use the principle of "many a little makes a mickle". In spite of their individually modest profitability, a lot of our minor investment projects may provide, on the nationally economy level, a fair contribution to economic progress.

It stands to reason that a physically more favourable location site is of preferential value compared to one less favourable. It is primarily this value that should be helped by engineering geology to getting realized. Consequently, the economic use of engineering geological research can be assessed even in quantitative terms, inasmuch as we are able to compare, in terms of value, the following relations:

1. The relation of engineering geological research to the cost of investment: engineering geology's share of the cost input.

2. The relation of the differential rent due to dissimilarities in the natural or physical characteristics of the relevant location sites of engineering structures planned /differences in the geotechnical characteristics/ to the investment cost: differential cost savings fraction of the investment cost.

3. The relation of the cost of engineering geological research to the differential rent according to paragraph 2: research efficiency index.

In connection with engineering geology's share of the cost input again, the local investment projects represent a special case. Namely, the research cost item in these may, in magnitude, come already quite close /mainly more scrutinized on-the-spot tests, e.g. test loading, etc./ to the investment cost input and it may reach the level of operation cost. In big projects, of course this is not the case.

It would be theoretically more useful to compare the research cost to the total output of the resulting plant during the whole span of time of its operation. Instead of using the research cost fraction to this purpose. Notably, by select-

ing a more advantageous location site, engineering geological research does contribute to a prolongation of the total life span of the plant. Consequently, the ratio of the total output of the enterprise in question and the cost of engineering geological research too will improve. However, only the most advanced industrial nations are able to give information on comparisons of this kind by the example of their industrial plants which now in many places can be regarded as industrial monuments, but even these are very few in number. The same holds true for the relations of paragraphs 2 and 3. These conditions are illustrated in Fig. 3 /after P. Kozma, the famous Hungarian economist/.

"LYFE CYCLE"

of "ENGINEERING-GEOLICAL" OBJECTS
inputs and outputs

Fig. 3.

Biggest problem in the developing countries is faced in acquiring the sources for B. If the is no rent, the sales returns curve will be more flat, the amortization period will be prolonged, i.e. the rent will accelerate the amortization and increase the profitability.

From the national economic viewpoint, the rent is a macro-level efficiency factor. The greater the number of engineering projects able to maximize the use of the differential rent through engineering geological efforts in a country, the
The greater the quantity of the material resources a particular economy can use for other arbitrary purposes, i.e. for structuralizing the national economy. In my opinion, this relationship is that which expresses the usefulness of engineering geology for the developing countries. Namely, it is very important for them to be able to exploit their own resources with such an intensity that will enable an accumulation of funds as a result of the activities. Constituting a growing percentage of the net social income, the regroupable income from rents may contribute to the implementation of their public welfare and social programs, to a proportionate development of their economy.

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ABSTRACT

Engineering Geology and Geotechnology—the newly developing border-land discipline of applied earth-science and engineering technology-offer solutions to problems related to the natural environment of engineering structures. Recent impetus to the growth of Geotechnology has been provided by Man's increasing concern with the preservation and optimum utilisation of his environment.

In India, as elsewhere in the World, the early growth of engineering geology was related to the construction of canal systems (18th Century) and communication routes (19th Century). Its later growth in the 20th Century was related to dam and reservoir construction; to military needs; to the requirements of nuclear power generation; and to the development of National economies after the War, through the utilisation of water resources for irrigation and power generation. The growth and development of Engineering Geology in India is reflected by the growth and development of the Engineering Geology and Ground-water Division of the Geological Survey of India, which, initiated in 1945 with only 6 geologists, increased in strength to 120 geologists in 1981, when it handled about 500 items of investigation per year. Nine separate directorates of Engineering Geology operate now from six Regional Centres of the G.S.I. and their growth has been in direct response to the developmental needs. Environmental Geology has also been given a fillip during the decade 1970-80 and in 1981 six Directorates were operating from the six Regional Centres of the G.S.I.

The methods, techniques and tools of engineering geological and environmental geological investigations have vastly improved, both in scope and in variety during the period 1950-80. Standardisation of the methodology and techniques of geotechnical investigations of river valley projects has also been effected during this period. The facilities for teaching and research in engineering geology and geotechnology, up to the Post Graduate level, now exist in 10 of the roughly 40 Universities and Institutions offering courses in Geology. The other important milestones of progress relate to the inception of the Indian Society of Engineering Geology in 1965; its sustained growth and scientific contributions during the last 15 years and the development of the International Association of Engineering Geology, first conceived in India at the time of I.G.C. in 1964.

India's achievements during the last 30 years reflect the significant advances that have taken place in the areas of engineering geology and geotechnology, which had indeed permitted the successful completion of high concrete dams on soft rock; of high rock-fill dams with extensive drainage facilities in soft-rock abutments; of tunnels through squeezing, flowing and methane gas-filled ground; of chemical grouting of poor foundations; of construction of underground power houses in very difficult rock media with multiple sets of rockmass discontinuities; and of impoundment of reservoirs on cavernous

*The French version of the abstract appears at the end of the paper.
limestones. Future challenges are equally difficult and can be faced with the available trained Indian expertise very effectively, if this can be reinforced by the induction and imbibement of newer techniques, methods and concepts of assessment of environmental conditions, available in specific areas of the world.

I. INTRODUCTION

It is well known that the applied geoscientific discipline of Engineering Geology deals with the assessment and the solution of geological problems that arise in the course of the planning, design, construction and maintenance of civil engineering structures. The definition of the applied geoscientific field of Geotechnology is not as widely understood. About 3-4 decades ago, the terminology of Geotechnics or Geotechnology was introduced, so as to encompass the newly-developing cosmopolitan and comprehensive, borderland discipline of applied science and technology, which represents the integration of elements of Civil Engineering Technology with the data of not only the Geological Science, as is the case with Engineering Geology, but also of all the other Earth Science disciplines, like Pedology, Hydrogeology, Soil Mechanics, Geophysics, Seismology and Geosismology, so as to provide a framework of information that can offer solutions to problems connected with the natural environment of engineering structures. In the present review, while devoting attention, primarily to the history and development of Engineering Geology in India, the related applications, involving all the other Earth Science disciplines have only been briefly stated. Summarised accounts of the applications of all the Earth Science disciplines in the course of the planning, design and construction phases of several Indian Projects for River Valley development have already been presented elsewhere (Krishnaswamy, 1972). The basic compulsions for the growth of Geotechnology have been the endeavours of Man towards the planned utilisation and conservation of Natural resources and, lately, towards the preservation and optimum development of Man's environment. The increased activities of Man, resulting in substantial industrialisation and urbanisation of his environment have sparked off further growth of Geotechnology, through increasing studies of the environmental aspects in the local, semi-regional and regional scales.

Elsewhere in the World, the early growth of Engineering Geology was related to the development of water-supply canals in the late 18th Century and, to the development of transport facilities in the 19th Century. This pattern of growth has been repeated in India as well. In early 20th Century, special studies and advancements were made in the field of dam and reservoir geology and, India had also taken advantage of all these advancements in the course of its own applications in the field of Geotechnology. In the 20th Century, the requirements of military warfare led to special methods of terrain evaluation in the other parts of the World and at the same time, in India, a branch of Military Geology was also developed in the Country's National Geological Survey organisation.

In the post-War period, the requirements of the Atomic Age lead to developments in the criteria for the selection of sites for atomic power plants; in the study of the effects of atomic blasts; and in the design of underground caverns for blast-resistant structures as well as for the storage of radio-active wastes. Limited applications have also been developed in India in some of the above-mentioned areas of activity relating to the Atomic Age.

The most significant developments in the field of Geotechnology, in the Post-War period, however, relate to the major expansions that took place in the areas of water and power development, which were dictated by the needs of reconstruction of the war-torn economies; by the expansion of the economies of a number of industrially advanced countries; and by the needs of the developing economies of the newly independent countries, like India.

During the decade 1970-80 the impact of Man's activities on the environment and, the need to preserve the same for deriving its optimum potential have generated significant lines of study in the realm of Environmental Geology and, India too had followed this lead by initiating environmental studies of urban areas, to start with, which have now been extended to cover rural areas as well as different types of geomorphological environments.

All these advancements in the field of Engineering Geology and Geotechnology can be analysed in three phases of development, in so far as India is concerned, viz., the First Phase covering the period 1850-1900; the Second Phase, encompassing
the period 1900-1950 and the Third Phase covering the period 1950-1980. This review will be particularly concerned with the panorama of progress in India, as recorded during the last 130 years. Comparisons of progress made in India with the progress recorded in Europe and America have been presented earlier by the author (Krishnaswamy, 1974). The history and development of Engineering Geology in the Geological Survey of India has been reviewed by Balasundaram and Rao in 1972, along with the details of the engineering projects handled by the G.S.I. in the papers contributed by B.M. Hukku et al; P.B. Srinivasan et al; B. Ramachandran et al; and S. Ray et al, appearing in the Records of GSI, V. 104(2) Interested readers can get from the cited reference, more information on the technological aspects of engineering geological developments in India.

II. THE FIRST PHASE OF DEVELOPMENT

In India, the earliest known application of geotechnology to the construction of buildings is the Taj Mahal, built during the period 1632-1650, in which the principle of transfer of load to depth, using piles, has been successfully utilised. Thus, this monument has been built on cylindrical well foundations sunk into the soil at close intervals. The soundess of the design is testified by the fact that even after three centuries of its existence, the lines and angles of the structure are as accurate as first produced (Legget, 1972). However, the builders of the 17th and 18th centuries were guided more by the empirical knowledge and experience gained and passed down from generation to generation than by the appreciation of the physical laws of Nature, or, by the related scientific observations and inferences. Systematized application of the knowledge of geological principles and processes to the construction of engineering structures came up in India only in the 19th Century.

India was one of the few countries in the World that had appreciated the value of applying geological principles to engineering problems, as early as in the second half of the 19th Century. In the case of application of geology to the selection of dam-sites, this had happened about 50 years earlier than in the Western nations. However, the Indian applications of the knowledge of the Earth Sciences were not on a continuing basis, and were necessarily on an ad hoc basis, the applications being demand-based. A systematic build-up of knowledge and techniques of engineering geology, with consistent and continuing applications, leading to very specialised expertise, took place much later in India, in the Second and Third Phases of development, which are discussed later on in this review.

Thomas Oldham of the Geological Survey of India is considered to be the pioneering engineering geologist of the 19th Century. He had analysed, as early as in 1852, the geological factors influencing the choice of the proposed railway alignment between Calcutta and Patna, a distance of 500 kms. The next effective application of geological knowledge to engineering problems was by Sir Thomas Holland in 1884, when he had analysed the causes and effects of the disastrous landslide that took place in the Birighi Ganga Valley, located in the Middle Himalayan zone of the State of Uttar Pradesh. This landslide had formed a 300 m. high dam with a base-width of some 2 to 3 kilometres along the Birighi Ganga river and impounded the river flow to form the Gohna Lake. The lake created by the dam survived for 86 years, when the natural processes of silting, as well as the series of landslides that had taken place in the same valley in 1970 contributed to its disappearance.

With great scientific acumen, Sir Thomas Holland had analysed the stability of the landslide-dam in terms of the dimensions of the dam and the thrust of the mass of water stored behind it and predicted that only a small portion of the dam, near its crest will be washed away, when the lake got filled up to its maximum capacity, about a year later. He had also forecast that when this event took place, a residual lake of predicted dimensions will remain. The Gohna landslide dam, as predicted by Holland, did give way in the crestal portion alone and this too within a week of the time he had forecast for the event; the Gohna lake also survived with the dimensions as predicted by Holland. What is of greater significance is the fact that on the basis of Holland's predictions, adequate precautions had been taken by the civil administration of the area, and as a result, not a single life was lost, due to the floods caused by the partial dam-failure, although the height of the flood-wave was as much as 60 metres at the town of Chamoli, situated downstream of the confluence of the Birehi Ganga with the main-stem-river, Alaknanda, and was 20 metres high close to the populated town of Hardwar, where the main-stem river, now known as the Ganga, debouches to the plains from the Himalayan ranges.

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In the 19th Century, major engineering projects for water storage and utilisation were constructed in India and, in the construction of all these structures, geological principles were taken into consideration by the Construction engineers. (Balasundaram and Rao, 1972). A striking example of these structures is the Ganga Canal System in Northern India which was completed in 1854. The Godavari and Krishna Canal Systems in Southern India were constructed in 1855 and, in the case of the Godavari System, for the low diversion dam at Dowaleswaram, located on the highly permeable foundations of the Godavari river bed, an ingenious partial cut-off to lengthen the path of flow was provided, by using staggered open wells, filled with compacted clay. This geotechnical innovation anticipated the developments that took place 50 years later in the field of control of leakage through permeable foundations, using Dr. A.N. Khosla's hydraulic theory of design of weirs on such foundations propagated in 1934.

The first dam for storage of water for city water-supply was completed up to a height of 30 m. at Khadakvasla near Poona in 1869. Around the 1870's, the first inter-basin diversion of river flows was accomplished through the construction of the Periyar Reservoir in the then Travancore State (now part of Kerala State) which diverted the flows of the westward-draining Periyar river to the east flowing river system in the adjoining Madras State (now known as Tamil Nadu State).

During the years 1888 to 1889, the Geological Survey of India reported upon the suitability of two dam-sites in the then States of Mysore and Madras (now Karnataka and Tamil Nadu). The first hydro-electric power station in India was completed at Simla in 1897. During the period 1898-99, several dam-sites in the then state of Madras were investigated by the Geologists of the Geological Survey of India. The landslides of Nainital, the well-known Hill Station in the Himalaya of Uttar Pradesh, were analysed in terms of the planned building activity, as early as in 1895-96. In 1897 R.D. Oldham came out with his classic Memoir on the Great Assam Earthquake of that year, which, for the first time, put forward the concepts of different types of Earthquake waves (P and S) and thus laid the foundations of Modern Seismology.

III THE SECOND PHASE OF DEVELOPMENT

During the first 47 years of this phase of development (also called the pre-Independence Period) the Geological Survey of India had carried out geological investigations of as many as 105 dam and reservoir sites intended for irrigation and flood control; 6 investigations for railway alignments connecting the then Provinces and Bihar; Bombay and Sindh; 3 investigations of bridge-sites and 5 investigations of landslides, of which the continuation of the studies at Nainital got special attention.

In the pre-Independence era, the Geological Survey was also responsible for the engineering geological studies required in Burma, Baluchistan and North-west Frontier Province (the last two forming part of Pakistan now) and, in this connection, had investigated 36 dams (including hydro-electric schemes); 6 bridge-sites and two railway alignments in the cited erstwhile territories of the British Empire.

The brilliant dissertations by the geologists of the G.S.I. working on the later Assam Earthquakes; on the Kangra Earthquake of 1904 and the Bihar-Nepal Earthquake of 1934 helped to further advance the discipline of geoseismology in India and to propagate the concept of evaluation of seismic hazards, based on regional and local geology. In the years 1944-45, the technique of assessment of status of activity of faults by matching and surveying the levels the river-terraces straddling the trace of faults was first introduced.

The second Great World War (1939-45) saw the creation of the Strategic Branch of the Geological Survey which concerned itself with the selection and construction of aerodromes; the choice of alignments for roads; selection of construction materials; identification of sites for landing on beaches and suggesting locations for effecting water-supply to the military installations. After the termination of the War, the Strategic Branch was wound up only to give place, in 1945, to a Specialist Division, known as the Engineering Geology and Ground-Water Division. This Division, in comparison with the predecessor-Strategic Wing- had a much greater compass of applications of geological knowledge to engineering and ground-water projects of the Post-War India. The Division was started with a contingent of only 6 geologists, and in 1950, five years after its formation, its strength had increased to 18 geologists.

With the winning of political indepen-
dence in 1947, there arose the need generating economic independence and, an era of construction and development was initiated in the country. In this era of developmental activities, the Specialist Division of the G.S.I. dealing exclusively with engineering and ground-water developmental projects, gave a great helping hand. Thus, the Division met the requirements of geological advance on the newly emerging, multi-purpose river valley projects, like those of the Damodar Valley Corporation (D.V.C.)-modelled on the lines of the T.V.A. in U.S.A. - the Hirakund and the Bhakra Multipurpose Projects. The first whole-time Resident Geologist on multipurpose projects was posted at the site of the Hirakund Dam in Orissa State in 1947, for tendering day-to-day advice to the Project engineers, on the planning, design and construction aspects of the Project and, the author of this review had the distinction of filling up this post. Subsequently, similar Resident Geologists were posted at the sites of several other major river valley projects in the country like the D.V.C. and Bhakra Projects; in 1950, at the close of the Second Phase of Development, 12 such resident geologists functioned at several Project sites, which represented about 70% of the total strength of the Specialist Division of the G.S.I.

IV THE THIRD PHASE OF DEVELOPMENT

This phase has been the most fruitful one of all the three phases, from the point of view of the development of knowledge, methodology and techniques of engineering geology and geotechnology in India. This came about because, during this period, the whole gamut of developmental activities conceived under the different Plan periods was executed, commencing from 1951, the first year of the First Five Year Plan and ending with the second year of the Sixth Plan in 1981.

The Engineering Geology and Groundwater Division of the G.S.I. grew in strength in response to the needs of the development of the Country. In 1957, it was bifurcated into the Divisions A and B, just 12 years after its inception. The Division A was responsible for the developmental activities in the northern and eastern areas of the country with its Headquarters at Calcutta, while the Division B handled the requirements of the southern, central and western parts of the country with its Headquarters at Hyderabad. Both these Divisions handled, over a period of 2 years, the requirements of 282 Irrigation and Flood Control Projects; 51 Hydel Projects and 60 other items relating to Communication Routes, Hill-side Stability etc. (Balasundaram and Rao 1972).

Two years after the bifurcation of the E.G. & G.W. Divn., three separate Divisions were formed to deal with engineering geology alone. These were located at Lucknow (for the northern and western parts of the country); at Calcutta (for the eastern and the north-eastern parts of the country); and at Hyderabad (for the southern and central parts of the country). The author of the present review had the privilege of heading the Engineering Geology Division at Lucknow for a period of 11 years from its inception in 1959.

In 1967, two more Engineering Geology Divisions were created at the two, newly established Regional Centres of the G.S.I. viz., at Nagpur and Jaipur, respectively, for the central and western parts of the country. In 1967 the strength of the Engineering Geologists in the G.S.I. was only 66. The sixth Engineering Geology Division of the G.S.I. was started at Shillong during the Fifth Plan period, along with the establishment of a new Regional Centre of the G.S.I. for the North-eastern Region. During the Fifth Plan period, the Engineering Geology Divn. in the Northern Region was strengthened and was further bifurcated into E.G. Division - East and West and, later, trifurcated with the addition of the E.G. Division - Central.

After the commencement of the Sixth Plan (1981) the Enng. Geology Divisions in the Northern Region, dealing with complex problems in the Western Himalayan set-up were quadrupled, with the addition of the E.G. Division (South) so as to look after the geotechnical problems relating to the Peninsular geological set-up in the southern part of Uttar Pradesh, besides looking after the Geotechnical Laboratory that had been set up in the Region a decade earlier. Such Geotechnical Laboratories, are now functioning at all the six Regional Centres of the Geological Survey of India and are giving the required assistance in the laboratories to the Engineering Geologists working on different geotechnical problems all over the country. The strength of the engineering geologists in the G.S.I. which rose from 6 in 1945 to 66 in 1967, stood at about 120 in 1980-81, and these specialists are functioning now under 9 separate Directorates of Engineer-
ing Geology, four at Lucknow in the Northern Region and one each in the remaining five Regional Centres of the G.S.I. at Calcutta, Shillong, Hyderabad, Nagpur and Jaipur.

That the increase in the strength of the practising Engineering Geologists in the country was in response to the developmental needs will be explicit from the fact that as against 56 items of engineering geological investigations handled during 5 years prior to the launching of the First Five Year Plan in 1951, the number of items of investigations handled during the First Five Year Plan (1951-56) rose to 266; during the Second Plan (1956-61) to 393; and to 1030 items in the Third Plan (1961-66). In the second year of the Sixth Plan, during the Field Season 1981-82 alone, or in one year, as many as 515 items had been inscribed in the Programme of the G.S.I., or, roughly doubling of the workload, if the same trend were to continue in the remaining years of the VIIth Plan period, as compared to the position in the Third Plan. In general about 60% of the investigations handled by the engineering geologists of the Survey relate to irrigation, Flood Control and Power Projects while the remaining 40% relate to Communication Routes, Stability of Slopes; Building Sites; Construction Materials, Earthquake effects etc.

In comparison with the Second Phase, geophysical inputs in the early stages of investigation of river valley projects have grown in volume in the Third Phase, although for want of man-power and equipment, it has not been possible yet to deploy composite geological-geophysical teams on all the investigations of Project-sites handled by the Geological Survey of India, which, indeed would be a desirable objective, taking into account the trends of development in Geotechnology in the other parts of the World.

The application of Engineering Geology to serve the needs of environmental appreciation started in 1970 in the country, when a combined team of photogeologists, engineering geologists, hydrogeologists, geophysicists, engineers and soil-scientists, working under the overall coordination of the author, studied different aspects of the surface and subsurface geological environment of Hyderabad City from the point of view of the future growth-needs of the City. Later, an entire rural district (Anantapur district, Andhra Pradesh State) was encompassed by similar environmental studies, the results of which have been published in the Miscellaneous Bulletins No. 47 of the G.S.I. (1979). Many other rural districts of the country; some of the deltaic, coastal and desert areas; industrial centres; and semi-urban areas have all been taken up subsequently for environmental appreciation and resource assessment for optimum utilisation of the environmental potential, by the different Regional Centres of the G.S.I., as well as by geoscientists of some of the Universities and Institutions. The G.S.I.'s endeavours have been summarised in the publication entitled "A Decade of Environmental Geology in India" released in 1980, on the occasion of the Benares Session of the Indian Science Congress. Indian Institute of Technology, Powai, Bombay, has dealt with, in a Special Publication brought out by it, the contributions made towards environmental appreciation and resource management of the Chandrapur District of Maharashtra State. While Environmental studies by the G.S.I. during the Fifth Plan Period were carried out by the Division of Quaternary Geology, Geomorphology and Environmental Geology, separate Divisions for dealing with Environmental Geology alone, have been created after the commencement of the Sixth Plan, in 1981, at all the six Regional Centres of the G.S.I. although each with a small group of geoscientists. It is hoped that with this nucleus for growth, significant advances in the area of environmental appreciation and management will be made by Indian geoscientists during the remaining 20 years of this Century.

The methods of engineering geological and environmental geological investigations have also vastly improved during the Third Phase of development, with satellite-generated and photogeological data inputs. In the area of Engineering Geology, while, in the earlier phase, geological advice was largely based on the geological maps on 1:50,000 scale (or equivalent 1" to 1 Mile) and limited field studies, aided by calyx-drill hole data and few laboratory tests on rocks cores, during the 3rd Phase, detailedgeological mapping of Project sites and Project areas on scales ranging from 1:15,000 up to 1:100; geophysical inputs by seismic and resistivity survey methods; seismo-tectonic evaluations; extensive diamond core drilling with water percolation tests; extensive tunneling into the abutments for facilitating the observation of in-situ conditions; exploration through 90-centimeter diameter holes; in-situ field tests; and vast improvements in the presentation of core drilling data and tunneling data in detailed logs, often in three-dimension,
have given a wide-base data support to the engineering geologist, thus enabling them to derive well reasoned conclusions on seismicity and site competency; to demarcate weaker features and to suggest the broad pattern of remedial treatment.

The third phase of development of Engineering Geology in the country is also significant from the point of view of the growth of facilities for teaching and research in the fields of Engineering Geology and Geotechnology. Notwithstanding the availability of good books on Engineering Geology, like Cyril S. Fox's, at the time of Independence, very limited facilities were available and, this too, only at a few of the Universities, for imbuing principles and practices of Engineering Geology. However, at the end of the first decade of the Third Phase of Development, in 1961, there were about 30 universities and Institutions of Higher Learning, teaching Geology up to the Post-Graduate level (i.e., up to M.Sc or B.Tech.). Of these, about 6 of them were offering a special course in Engineering Geology, as a part of the total curriculum of Applied Geology. In 1981, out of the nearly 40 Universities teaching Geology up to the Post-Graduate level about 10 Universities were offering special courses in Engineering Geology/Applied Geology and, facilities now exist for research at Post-Graduate level, in the fields of engineering geology, hydrogeology, soil mechanics, geoseismology, rock mechanics, and foundation treatment. The Indian School of Mines and Applied Geology, founded as early as in 1926, introduced in 1981, a special post-Graduate course in Engineering Geology.

During the 3rd phase of development of Engineering Geology, the Indian Standards Institution made important contributions to growth through standardisation of methodology and techniques of investigation of river valley projects, utilising the consolidated knowledge and expertise available with the Geological Survey of India, Universities and Institutions and Irrigation, Flood Control and Hydro-electric Organisations at the State and the Central levels. Notable amongst these contributions to the growth of engineering geology and geotechnology are several Standards and Codes of Practices dealing with different techniques of sub-surface exploration: collection and presentation of drilling information; geological mapping and symbolisation; methods of foundation evaluation and preparation of a Seismic Zonation Map with accompanying codes for seismic resistant design and construction of engineering structures.

Another important milestone along the road of development of Engineering Geology in the Country is the establishment of the Professional Society of Engineering Geologists at Calcutta in October, 1965. Starting from very humble beginning the Society has grown in strength and, the current membership of the Society stands at 500, which includes geoscientists and engineers interested in the development of engineering geology and of which more than half the number are in the category of Life Members. The Indian Society of Engineering Geology has held several Symposia and Seminars in the fields of Engineering Geology and Geotechnology during the last 15 years of its existence; notable amongst them being those devoted to i) Geological and Engineering Problems of River Valley Project (1965); ii) Rock Mechanics (1968); iii) Koyna Earthquake of 1967 and related problems (1968); Geological and Engineering problems of Tunnelling (1973); Landslides and Toe-erosion problems with special reference to the Himalayan Region (1975) and Three Decades of development in Engineering Geology in India (jointly with G.S.I.) in 1981.

Keeping pace with the development of the National Group of Engineering Geologists, is the birth of the International Association of Engineering Geology, which took place in 1964 at New Delhi, India, at the time of the session of the International Congress of Geology. A number of geoscientists who had gathered for the Congress from several countries of the World and who were interested in the development of Engineering Geology, joined hands with the members of the Geological Survey of India and Professors of Indian Universities and Institutions teaching Engineering Geology and decided to form the International Association of Engineering Geology. It is indeed gratifying to several members of the National Group of Engineering Geology in India, who are also members of the International Association, that after 18 years after its birth, the International Association is coming back to the Mother Country, on the occasion of the IVth Congress on Engineering Geology.

V REVIEW OF ACHIEVEMENTS

It will now be appropriate to take stock of some of the achievements in the field of engineering Geology and Geotechnology in India, particularly during the Third Phase, as during this Phase alone, the country had seen the culmination of
great Civil Engineering enterprises, where the cited geoscientific disciplines had played very notable part in ensuring their successful completion. In a review of this type, it is not possible to give a complete list of achievements, covering every part of India. The list which follows can at best be termed as representative of the achievements and, in no sense, a complete or comprehensive one.

The Indian achievements during the Third Phase include the successful completion of i) the longest dam in the World at Hirakud, Orissa, State, with 5 kilometre-long main dam made of concrete/masonry/earth and about 20 kilometres of low earthen dykes to form the reservoir; ii) one of the very high straight-gravity concrete dams in the world at Bhakra, Himachal Pradesh State, founded on soft rocks and medium hard rocks which were riddled with fault zones cutting across the foundations and abutments in different attitudes; iii) one of the fairly high Cyclopene concrete dams in the world at Koyna, Maharaashtra State, which survived a major earthquake that originated very close to its location; iv) one of the high concrete arch dams in the world at Idikki, Kerala State, founded on near ideal (Archaean) foundation rocks, with an equally ideal gorge-setting for this type of dam; v) the highest stone-masonry dam in the world at Nagarjunasagar, Andhra Pradesh State, straddling across the contact of Archaean and Pre-Cambrian sedimentary rocks; vi) one of the earliest earthen-cum masonry dams in India at Ukai, Gujarat State, here chemical/clay grouting was done for the treatment of a major fault zone; vii) one of the highest rock-fill dams in India with extensive drainage facilities in the soft-rock abutments at Ramganga, Uttar Pradesh State; ix) one of the large-diameter, soft-rock tunneling projects in India at Pong, Himachal Pradesh; x) one of the longest inter-river diversions in the World, the Beas-Sutlej Link Project, Himachal Pradesh, with 25 kilometres of tunneling, some of which had to negotiate very difficult flowing and squeezing ground; xi) one of the most difficult underground power-house in the Himalayan setting, with several sets of discontinuities in the surrounding rock mass which had to be stabilised by continuous roof support and tied with prestressed cables at Chhibro, Yamuna Hydel Scheme, Stage II, Phase I, Uttar Pradesh State; xii) one of the few tunnels in the World through tectonically active ground encompassing two major Himalayan thrust-faults and some later tear-faults which necessitated triplication of a single tunnel, as at Yamuna Hydel Scheme, Stage II, Phase II, U.P. State and (xiii) with squeezing pressures that reduced tunnel diameter, at Giri Hydel Scheme, H.P. State; xiv) with methane-gas filled rock media besides squeezing pressures at Lok Tak Hydel Scheme, Manipur State; and xv) one of the successful reservoirs on fairly cavernous pre-Cambrian limestones as at Obra, Uttar Pradesh State. Besides the achievements on completed projects as about, challenging tasks in facing severe environmental conditions are currently in hand on the Kopili Hydel Scheme, Assam State, with elaborate grouting measures as contemplated for the highly cavernous Eocene lime stones and in preventing the loss of flux-grade pre-Cambrian limestones through submergence of the Bansagar reservoir in Madhya Pradesh.

Space does not permit of the detailed description of all the technological problems encountered on the Projects listed above and of the geotechnical innovations made to overcome these problems on the basis of the support given through the application of Engineering Geology and Geotechnology and by working in close collaboration with the Project design and construction engineers which, in fact, has been the hallmark of much of the engineering developments in India during the Third Phase of development. Briefly summarised, the geotechnical innovations could be listed as: i) extensive and intensive remedial treatment accorded to steep dipping beds of clay and shale and to fault zones by the Shasta Formula and variants of the same all involving transfer of load to the adjacent sounder members by beam action and by shear strength of the rock-concrete contact; ii) utilisation of the deformation modulus values and the natural stress fields for the evaluation of foundation response and for cavern stabilisation; iii) adoption of prestressed cable support and provision of concrete tunnel plugs by mining methods, to guard against sliding in the foundations and abutments; iv) novel construction and design practices while going through heavily flowing ground; tectonically active ground and methane-gas filled ground; v) chemical injections for treatment of fault zones and vi) use of cast-in-place concrete diaphragm in pervious media to serve as cut-offs under earth dams and vii) stabilisation of slopes against landslides by providing deep anchorage to the rock masses through reinforced concrete struts, besides the commonly adopted rock-bolting and shotcreting techniques.

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It can safely be said that with the knowledge and confidence gained in the Second and Third Phases of development of Engineering Geology and Geotechnology in India, almost all the engineering geological problems that are currently in hand or those that can be anticipated in the admittedly very difficult Himalayan setting and the less complex Peninsular setting, can be successively resolved with the help of indigenous expertise. However, the author feels that in the remaining 20 years of the Century, there will be need to induct modern techniques of subsurface exploration, using borehole geophysics and borehole photography; to imbibe the technology related to geological modelling and large-scale in-situ testing of rock and soil masses; to master techniques of monitoring of rock movements through instrumentation; to detect rock weaknesses ahead of tunnelling and to absorb ideas relating to assessment of rock quality for deciding upon the use of rapid tunnelling devices like the mole. Such induction of modern techniques and methods of assessment of environmental conditions, it is felt, will greatly reinforce the high quality of Indian expertise as is currently available in the areas of engineering geology and geotechnology, so as to resolve with renewed confidence and with speed to match the needs of a developing country all the challenging problems that lie ahead.

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ABSTRAIT

Technique Géologique de l'Ingénieur et Géotechnologie - la discipline de la marche nouvellement développée de science de terre appliquée et la technologie de l'art d'ingénieur offrent des solutions aux problèmes reliés à l'environnement naturel des structures de la technique d'ingénieur. Des impulsions récentes au développement de géotechnologie a été fourni par l'intérêt améliorant de l'Homme avec la préservation et l'utilisation optimum de son environnement.

En Inde, comme aux autres part du monde, le développement matinal de géologie de l'ingénieur était relié à la construction des systèmes canaux (18ème siècle) et communication routière (19ème siècle). Son développement récent dans la 20ème siècle était relié à la construction des barrages et des réservoirs et des exigences militaires, à l'exigence de la génération de l'énergie nucléaire et à développement de l'économie nationale après la guerre par l'utilisation des ressources d'eau pour irrigation et génération d'énergie. L'extension et développement de la technique géologique d'ingénieur et la division de la nappe de l'investigation géologique de l'Inde, qui a commencé en 1945 avec seulement 6 géologues, a augmenté sa force à 120 géologues en 1981, quand il a manipulé à peu près 500 items ou investigation chaque année. Neuf conseil d'administration séparées de Géologie d'ingénieur fonctionnent maintenant de six Centres Régionaux de l'Investigation Géologique de l'Inde et leur développement ont été en réponse directe à l'exigence développement. Géologie d'environnementale aussi a été donnée un coup de fouet pendant la décennie 1970-80 et en 1981 six conseils d'administration fonctionnaient de six Centres Régionaux de l'Investigation Géologique de l'Inde.

Les méthodes, des techniques et des outils de la technique géologique d'ingénieur et des investigations géologiques d'environnementales ont progressé immensément, en étendue et en variété pendant la période 1950-80. Unification de la méthodologie et des techniques des investigations géotechniques des projets des vallées des rivières ont été aussi effectuées pendant cette période. Les facilités pour enseignement et des recherches en géologie d'ingénieur et géotechnologie, jusqu'à niveau poursuivi après l'acquisition des titres universitaires, maintenant existe en 10 de à peu près 40 universités et des Institutions présentant des cours en Géologie. Les autres événements importants de progressés ont rapport à l'inception de la Société Indienne de Géologie d'ingénieur en 1965, son développement soutenu et des contributions scientifiques pendant les 15 dernières années et le développement de l'Association Internationale de géologie d'ingénieur, pour la première fois conçu en Inde au moment de I.G.C. en 1964. L'accomplissement de l'Inde pendant les 30 dernières années reflète des progrès significatifs qui ont pris place dans les domaines de géologie d'ingénieur et ceux de géotechnologie qui a varient permet l'achèvement avec succès de hauts barrage à béton sur roche molle, de hauts barrage roche-rempli avec des facilités étendues de système d'égouts dans les abords rochomolle de tunnels par compression, pays ruisseulant de gaz methane; de jointement au mortier liquide chimique de pauvre fondation; de construction des usine s génératrices souterraines dans les milieux de roche difficiles avec des séries multiples des discontinuités de masse de roche, et d'endiguement des réservoirs sur des roches calcaires caverneuses. Des défis futurs sont également difficiles et on peut les confronter avec des expertises indiennes disponibles très efficacement, si on peut le renforcer par l'induction et l'absorption des techniques plus nouvelles, des méthodes et des concepts d'évaluation des conditions d'environnementales, disponibles aux régions spécifiques du monde.
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