09 May 1984, 9:00 am - 12:00 pm

Research on the Permeability and Earthquake Damage of an Earth Dam Foundation

Zhi-Jie Zhu
Institute of Geophysics Academia Sinica, Beijing, China

Wen-Kai Liu
Exploration and Design Institute of Water Conservancy, nanjin, China

Research on the Permeability and Earthquake Damage of An Earth Dam Foundation

Zhi-Jie Zhu

Institute of Geophysics Academia Sinica, Beijing, China

Wen-Kai Liu

Exploration and Design Institute of Water Conservancy, Tianjin, China

SYNOPSIS The experiences on geological investigation, permeability test, prediction of seepage failure patterns and the earthquake damage of an earth dam foundation are presented in this paper. Basing on the monitoring data and seismic records observed from the seismic station on the dam, the prediction of reservoir induced earthquake and possibility of liquefaction are analysed.

1. GENERAL DESCRIPTION

The reservoir and dam are situated on an alluvial fan at the outlet of Hutuo River, North China (Fig.1) (Fig.2). The reservoir capacity is 1,000 million M^3. The main constructions content: a main earth dam on recent river bed, an auxilliary dam on the 2nd river terrace, a spillway and a small power station. In this paper only the problems of the auxiliary dam foundation is discussed. The auxiliary dam (hereafter "dam") is a homogeneous earth dam founded on the loose and complicated clay and sand gravel layer, with a maximum height of 19.2 M and a crest length 6970 M. At the beginning of the construction, owing to unsufficiency in geological materials, the engineers had been failed in their seepage control design. In the meantime the natural cohesive clay layer not far in front of the dam was excavated and used for the dam filling materials. Due to this incorrect treatment when the reservoir began to fill in 1961 with head only 5-8 M, serious piping and boiling phenomena had been found at the downstream toe of the dam. Then the water was forced to be discharged for repairing and strengthening the seepage control engineering works in order to reduce the seepage gradient. This supplementary engineering works include the elongation and thickening of the blanket in connecting with the impervious clay layer of the 1st river terrace, increasing and deepening of the relief wells along the dam down stream toe into the concentrated seepage belt of sand gravel layers. These further engineering treatment got very effective result. No piping phenomena were observed again in the last 20 years. During the large Xingtai earthquake($M=7.2$) of North China on 22, July, 1966, the seismic intensity at the dam site was 7^{0}. Some light earthquake damages were found on the spillway and the power station. These arose the attention of the danger of earthquake. In 1966 just after the large earthquake a strong seismic station was set up on the dam for observing the response of earthquake in order to get the necessary data for antiseismic design. In 1976-1977 a series penetration test was performed in the bore-holes in front and behind the dam for purpose to evaluate the liquefaction possibility of the dam foundation. It shows that only a few parts of the sand layer are qualified to be liquefied subjecting earthquake of 7^{0}, and owing to their deep seated location, if a good draining condition could be provided at the downstream toe, the danger of liquefaction could be out of consideration.

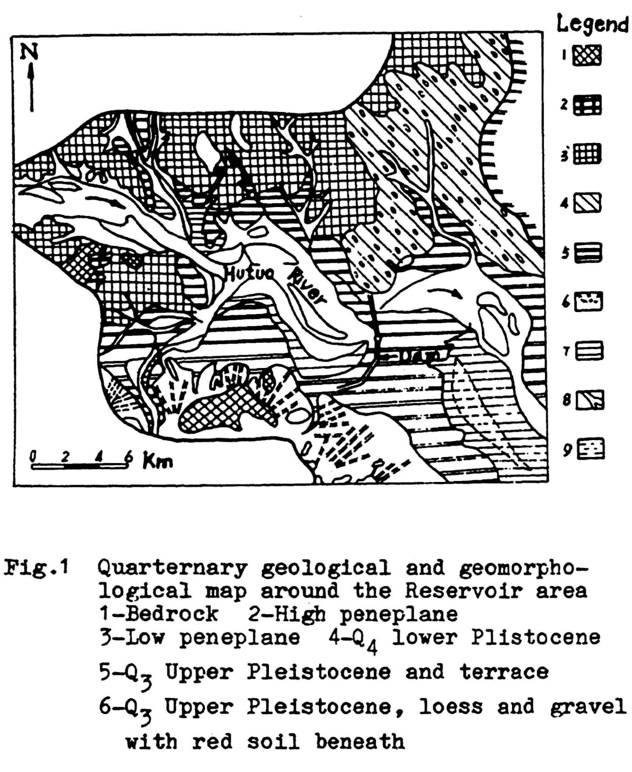


Fig.1 Quarternary geological and geomorphological map around the Reservoir area
1-Bedrock 2-High peneplane
3-Low peneplane 4-Q_4 lower Plistocene
5-Q_3 Upper Pleistocene and terrace
6-Q_3 Upper Pleistocene, loess and gravel with red soil beneath

7-Recent 1st terrace
8-Recent river bed
9-Down stream submerge area

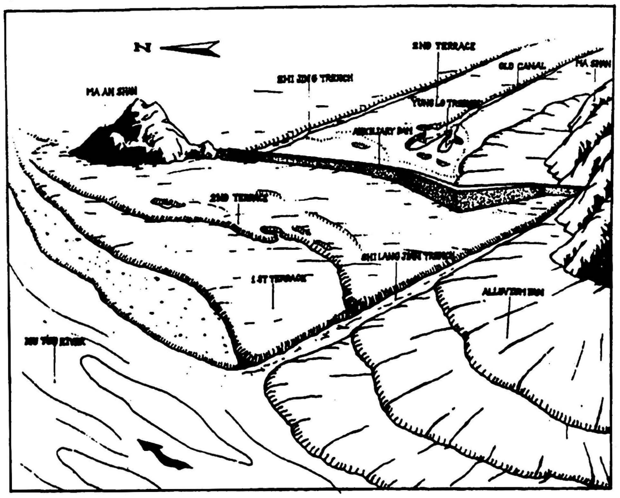


Fig.2 Panorama sketch of the auxiliary dam area

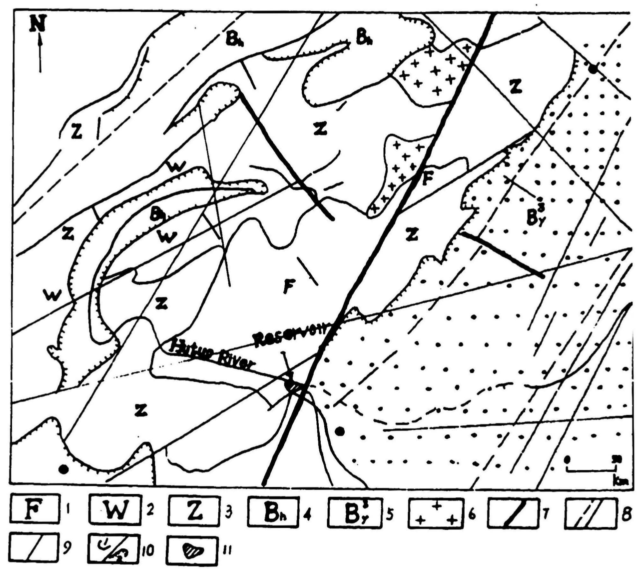


Fig.3 Brief tectonic map of the reservoir region
1-Fupingides 2-Wutaiides 3-Zhongtiaide 4-Late Yanshanian 5-Himalayan 6-Magmatism 7-Faults, Lithospheric Fractures, ascertained and conjectured 8-Faults, Crust Fractures ascertained and conjectured 9-Faults, Lineaments and fractures as shown on landsats photographs 10-Disconformities (1) and Unconformities (2) 11-Reservoir

2. ENGINEERING GEOLOGICAL CONDITION

The reservoir lies at the transitional belt between high mountain range and low alluvial plain, in the west is the Taihang mountaineous region, and to the east is the broad North China Plain, appearing a large tapography gradient. According to the analysis of the regional geological materials, most of the big regional fractures are striking NE, however, the strikes of the Mesozoic and Cenozoic enormous compressive faults are NNE, which are consistent with the streching direction of the mountain chain. Historical large earthquake always occured in this transitional belt along the NNE Fracture line.

The dam is located on the 2nd terrace of the old river bed of Hutuo river in the foothills of Taihang mountain range. The dam foundation is an extremely complex sand and gravel formation, 40M in depth and with a variable permeability over a wide range for different layers. Due to the negligence of the natural characteristics of this loose material foundation, engineers at the beginning of their design set up only a short upstream blanket and a few number of shallow relief wells at the downstream toe of the dam, which were considered to be quite enough for a dam only 20 M high. The construction of the project started in 1958, in 1961 after the occurence of piping phenomena during reservoir filling, engineers were aware that the dam was in danger.

Then the reservoir was forced to be emptied, and followed by a series of supplementary geological works and corresponding laboratory tests and in-situ tests, for the purpose to get detail descriptions of the loose layers beneath the dam. This supplementary investigation mainly consisted of two lines of bore-holes deeping into the bed rock in front and behind the dam, samples of different layers were taken from the bore-hole for laboratory tests. In the field a number of hydrogeological tests were carried out in the bore-hole to get the coefficients of permeability of different layers. All the above mentioned works are shown in Fig.4-a and 4-b.

From Fig.4, the bed-rock in the dam area are hard crystallised marble and phyllite of Pre-Cambrain age, lake shale deposit of Tertiary rests unconformably upon it. The old river bed deeply cut into the bed rock and accepted a thick Quarternary deposits, consist of four main layers. They are from top to bottom cohesive soil, sand, sand-gravel, sand-gravel and boulder respectively. (Fig.4-1), the average values of coefficient of permeability of different layers at its definite location got from the bore-hole subsurface water drawdown test are figured at the respective layers on Fig.4-1. From these data, it can be found there is a strong seepage belt between the upper part of the sand-gravel boulder layer and the lower part of sand-gravel layer, with coefficient of permeability ranging from 110-160 M/24 hours at the elevation approximately 75-95 M, lying at the middle portion of the profile (Fig.4-1) from pile number 1+100 to 4+300.

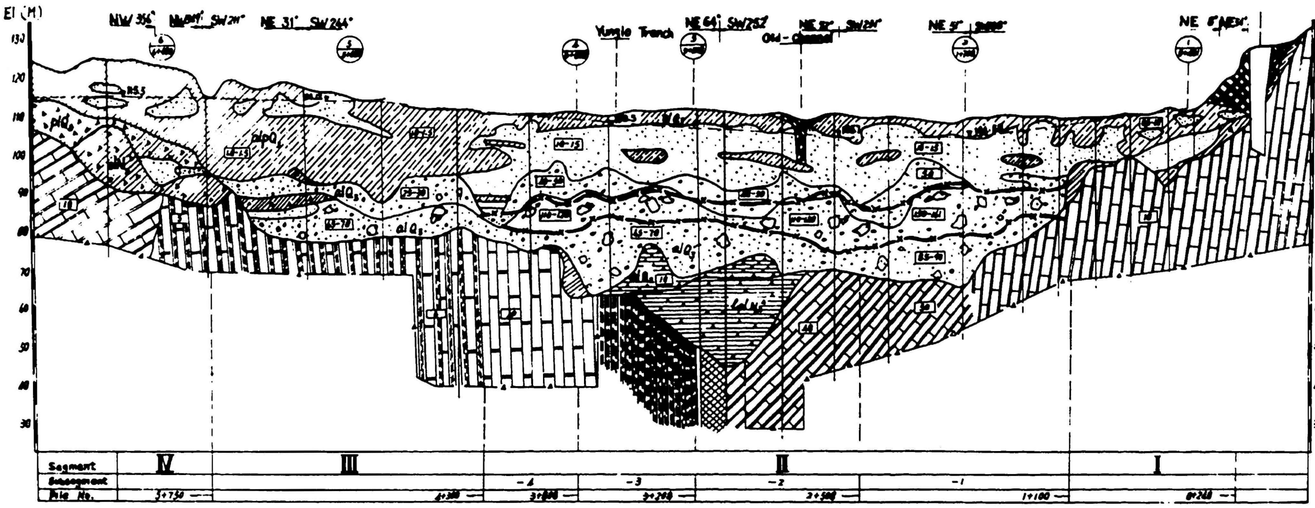


Fig. 4-2

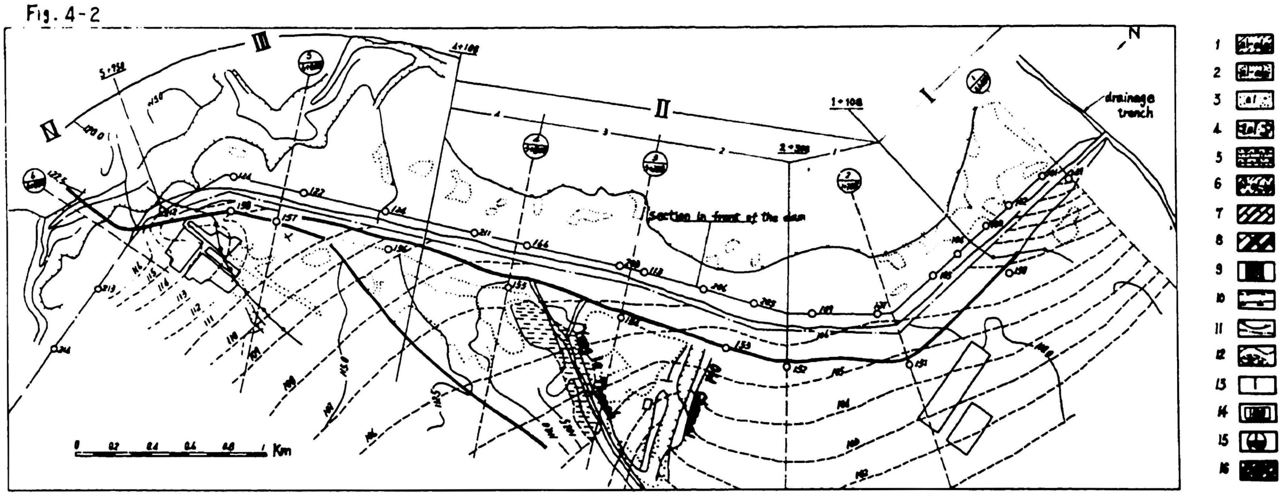


Fig.4-1 and 4-2
1-Upper pleistocene soil 2-Upper Pleistocene sand
3-Upper Pleistocene sand and gravel 4-Upper Pleistocene gravel 5-Middle Pleistocene conglomerate
6-M. and U. Pleistocene clay and breccia 7-Sinian siliceous limestone 8-Precambrian marble and phyllite
9-Fault and Fracture zone 10-Concentrated seepage belt and seepage lowest limit 11-2nd terrace and the boundary between sand and gravel 12-Contour line of underground water and its emerging area at reservoir level 118.0 M 13-Bore-hole 14-Coefficient of permeability (M/24 hours) 15-Number of the geological section and pile number
16-Filling soil

Fig.4-2 is the plane of the dam site, during the first filling of the reservoir, with head 5-8 M (E1.115-118 M), the piping and boiling phenomena occured predominently at the downstream toe between the pile number 2+308 to 3+808, where the old chennel and Yunglo trench are situated. In the profile, it is in consistent with the developing region of the concentrated seepage belt. While the water storage rose up to El. 118.0 M, water emerged in a certain area along the Youglo Trench at the downstream side.

The typical cross section of dam foundation is shown in figure 5 (pile number 3+208). In front of the dam, sand layer outcrops widly reaching the boundary of 1st terrace, forming an open path for the seepage flow through the concentrated seepage belt to the downstream side. Behind the dam, unfortunately a rather thick impervious cohesive soil layer lies upon the sand layer, forming a bad condition for the drainage of seepage flow. So it is obvious that only a short blanket and a small number of shallow relief wells could do nothing for the prevention of seepage flow, damage occured was a matter of course.

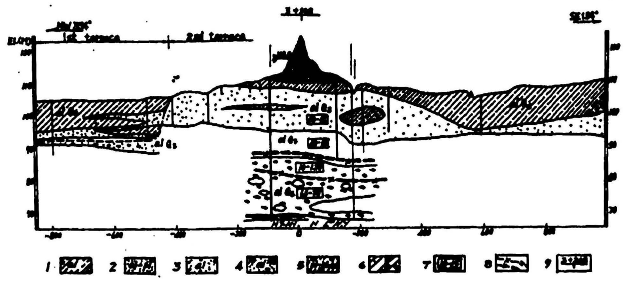


Fig.5 Geological cross section of the auxiliary dam at pile number 3+280
1-Soil 2-Sand 3-Sand-gravel 4-Sand-gravel and baulder 5-Clay and breccia 6-bed rock 7-Coefficient of permiability 8-Concentrated seepage belt

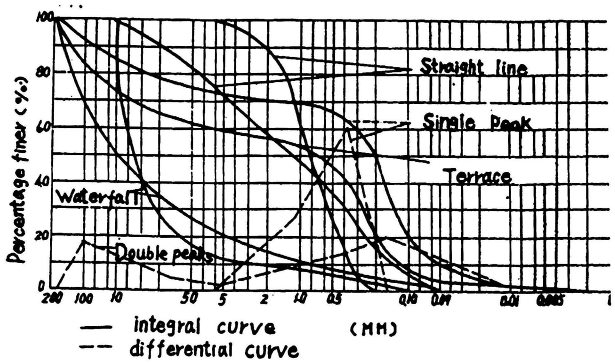


Fig.6 Integral and differential grain size distribution curves of the sand and gravel meterials showing the curve patterns

3. THE PROBLEM OF SEEPAGE STABILITY

3.1 Supplementary Laboratory Tests and Failure Patterns

According to theory and our experience, the seepage deformation patterns of loose sediments subjecting seepage flow would be soil flow for cohesive soil and sand with the uniformity coefficient η (d_{60}/d_{10}) less than 10;

soil flow or piping for sand and gravel , distinguished basing on the type of size distribution curve and the contents of fine grains (size smaller than 1 mm). In addition to the gradation analysis of a large amount of soil, sand and gravel specimens taken from the borehole of different layers, the piping tests were performed in a specially designed plastic cylindrical container, size 20 cm in diameter, in which the specimen of original particle size fraction were installed, the deformation and failure phenomena subjecting different water head were carefully observed and recorded.

The testing results are drawn into integral and differential size distribution curves, from which the deformation patterns can be distinguished as shown in Fig.6.

The integral distribution curves can be classified into three patterns, namely: straignht line, terrace and water fall. "Straight line" represents an uniform gradation, with a small ununiformity and more fine grain ($d < 1$ mm) content, the seepage gradient of which is comparatively higher; "Terrace" pattern in general being discontinuous in size distribution and lacking the intermediate particles, containing less fine grains and higher uniformity coefficient; "Waterfall" pattern contains more coarse grains with high ununiformity and low dry unit weight, seepage failure gradient being rather small subjecting water head. The possibility of piping can also be distinguished by the fine grain contents (p). It would be soil flow when $p > 80\%$; soil flow or piping when $25\% < p < 80\%$; piping when $p < 25\%$.

After the seepage deformation patterns are distinguished, further more the deformation developing trend needs to be predicted by studing the differential grain size distribution curves as shown in Fig.6. Curve of single peak represents "Safty type", while curve of double peaks belongs to "dangerous type".

3.2 Determination of Failure Gradient

Basing on the analysis of the characteristics of the sediments under the dam foundation, it can be pointed that the concentrated seepage belt plays the most important role in controlling the deformation and failure of the dam foundation, so in this paragraph regarding the failure gradient, only the failure gradient of the layers inner and outer the concentrated seepage belt will be discussed.

The concentrated seepage belt are located between the boundary of the sand-gravel layer and the sand-gravel and boulder layer. Accordin to the seepage condition, from top to bottom it can be divided into four subbelts, their deformation condition are different, shown in Fig.7 and Table I.

Table. I Theoretical analysis of the seepage deformation patterns inner and outer concentrated seepage belt

layer	Location	sets of test	d_{10}	d_{50}	d_{60}	η	Pattern of gradation curve	peak type	Type of seepage deformation	Failure gradient vertical	Failure gradient Horizontal
Sand-gravel	outer	7	0.18	12	22	122	terrace	Double		1.08	0.72
	inner	26	0.26	3.9	7.6	29.2	straight line	Single		1.00	0.66
Sand-gravel and boulder	inner	16	0.56	30.0	35.0	62.5	Water fall	Single	piping	0.15	0.10
	outer	5	0.25	45	65.0	260	Terrace	Double		0.94	0.63

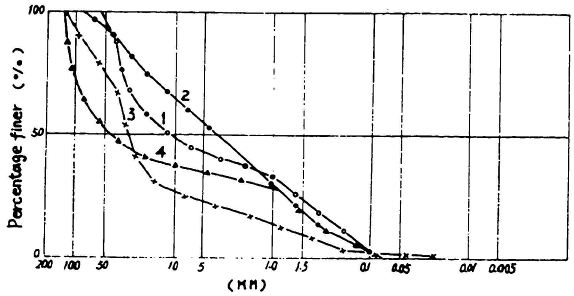


Fig.7 Average gradation curves of the sand-gravel layer, sand-gravel and boulder layer inner and outer the concentrated seepage belt
1-Outer concentrated seepage belt of sand-gravel layer
2-Inner concentrated seepage belt of sand-gravel layer
3-Inner concentrated seepage belt of sand-gravel and boulder layer
4-Outer concentrated seepage belt of sand-gravel and boulder layer

It can be noticed from the results obtained from theoretical analysis as shown in Table I, the deformation patterns of sand-gravel and boulder layer inner the concentrated seepage belt are all "piping", with a rather small failure gradient 0.1-0.15; while for the

Table II Analytical table of the seepage deformation of the sand-gravel, sand-gravel and boulder layers inner and outer of the concentrated seepage belt

layer	Concentrated Seepage belt	sets of laboratory test		Pattern of deformation		Failure gradient					
						theoretical		Laboratory test		Adopted	
		Theoretical	test	Theoretical	test	vertical	horizontal	vertical	horizontal	vertical	horizontal
Sand-gravel	Outer	7	3	Soil flow	Soil flow	1.08	0.72	0.81	0.54	0.95	0.63
	Inner	26	8	Soil	Soil flow or piping	1.0	0.66	0.94	0.63	0.94	0.63
Sand-gravel and boulder	Inner	16	2	piping	piping	0.15	0.1	0.18	0.12	0.18	0.12
	Outer	5	1	Soil flow	Soil flow	0.94	0.63	0.8	0.50	0.85	0.55

outer concentrated belt, the deformation patterns of the sand-gravel layer and the sand-gravel and boulder layer both are "soil flow". A part of sand-gravel layer inner the concentrated belt are "piping", while the others are "soil flow". The failure gradients for the above three subbelts being 0.63-1.08, all > 0.5. The size distribution curve shown in Fig.7 plotted according to the average values of the grain size analysis, which are in agreement with the results listed in Table I. By studying the geological condition, it can be found that the sand-gravel and boulder layer inner the concentrated seepage belt is situated in the central part of the main stream of the old river bed, with more coarse grains, higher permeability, forming relatively the unstable seepage layer. This macroscopic analysis is consistent with the results of laboratory test, as shown in Table II.

4. DYNAMIC FACTOR OF UNDERGROUND WATER

After the supplementary construction engineering works were accomplished during 1961-1963, the blanket was elongated to 100-400 M in connecting with the impermeable soil cover layer of the 1st river terrace, and the deep relief wells had been increased more than 200 in number. The dam underwent a serious examination of the big flood of 1963 with water level up to 121.79 M in the reservoir. It proved that the supplementary design and construction works are successfully done. From the following analysis of the underground water dynamic condition, the question why the seepage treatment has been in good performance can be easily answered.

The analysis of underground water dynamic factor is based on the long term underground water observation materials, as shown in Fig.8.

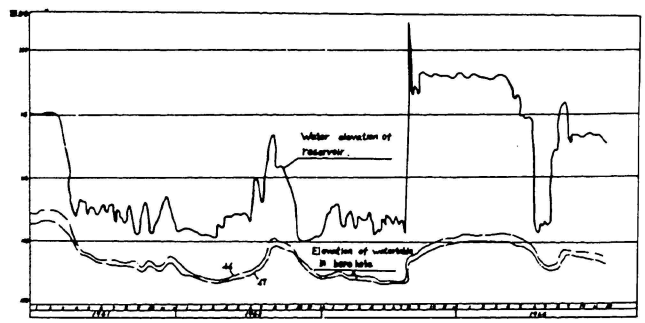


Fig.8 Process curves of the water level of reservoir and water table in the downstream observation bore-hole No.46 and No.47 during 1961-1964

From Fig.8 if we select the corresponding water level in reservoir and the water table in bore-hole 46 and 47 before and after the supplementary engineering works, it can be easily noticed that when the reservoir level at 118.0 M of February 1964 (post to supplementary engineering works) is taken to compare with the reservoir water level at 115.0 M of March 1961 (prior to supplementary engineering works), the water table in bore-hole 46 and 47 at that time had been lowered about 1-2 meters, revealed the supplementary engineering works had achieved good results.

In summarising the above descriptions, a short concluding remarks on the method of studying of seepage deformation of a loose sediment dam foundation may be drawn as the following sequence:

(1) Basing on the geological materials, macroscopically divide the various sedimentary

layers into different seepage segments;

(2) According to the test results of the physical and mechanical properties as well as the characteristics of the grain size gradation, theoretically distinguish the deformation patterns and determine the failure gradient as the following steps:

- (a) perform the laboratory test on grain size analysis, physical and mechanical properties of the sediment layers;
- (b) draw the integral and differential grain size distribution curves;
- (c) distinguish the deformation patterns by studying the integral grain size distribution curves;
- (d) according to the deformation patterns of different layers, select the appropriate equations to calculate the failure gradient (abbreviated in this paper);

(3) Determine the deformation patterns and practical failure gradient by carrying out the laboratory piping test;

(4) Correlate the results of distinguished and tested in connecting with the geological structure of the sediments, determine the deformation patterns and failure gradient;

(5) According to the subsurface water observation data in connecting with the result of electrical modelling test and theoretical calculation, calculate the failure gradient for different reservoir water levels (abbreviated in this paper);

(6) After consider the geological condition of the divided segments, characteristic of seepage deformation, method of testing and calculating equation, determine the safty factors of different deformation patterns of different layers, then compare with the real seepage gradient under the dam foundation and make out the evaluation of the seepage stability.

5. EARTHQUAKE AND LIQUEFACTION

5.1 Historic Earthquake and "Local Earthquake"

The reservoir and dam site are situated in the fracture zone of the piedmont of Taihang Mountain range. There are two groups of big faults running from NE and NNE respectively which are the tectonic background of the coocurence of earthquakes. Besides, the large topography gradient between the mountain region and the plane forms another favorable condition for earthquake. It is known from the records of historic earthquake information that from the year 1100 to 1966, there were 17 large earthquakes in shansi, and Hobei Province one of the largest was Xingtai earthquake occured on 22nd March 1966, the intensity of which around the reservoir was 7°.* The epicenters of the historic earthquake are shown in Fig.9.

It can be seen from Fig.9, earthquake(M=3-6) predominently occured in the zone from Xingtai to Shulu, concentrated along the NE direction, which coinsides with the strikes of the big faults, at a distance about 100-120 Km beyond the boundary of mountain and plane, only a few smaller earthquakes distributed not far in the NE of the reservoir around Lingshou . This phenomena can be supposed that the big faults are dipping SE with a certain dip angle, as the shocks happened some tens Km under the earth surface along the deep seated fault zone, the epicenters must be located toward east from the earth surface.

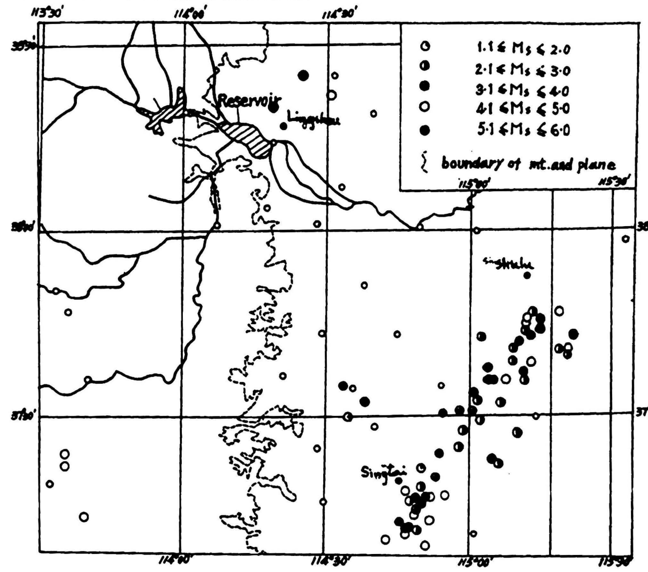


Fig.9 Distribution of epicenters of historic earthquakes

The epicenter of the diastrous large Xingtai earthquake (M=7.2) occured in 1966 is about 100 Km away from the reservoir, which had been lightly influenced the reservoir region and the dam site. The intensity was determined to be 7°. Some cracks were found on the concrete gravity dam of the water power station and the spillway, but the earth dam was harmless. However these light damages arose the engineer's attention, for they adopted 7° in their construction designs, in which only a little safety factor were left.

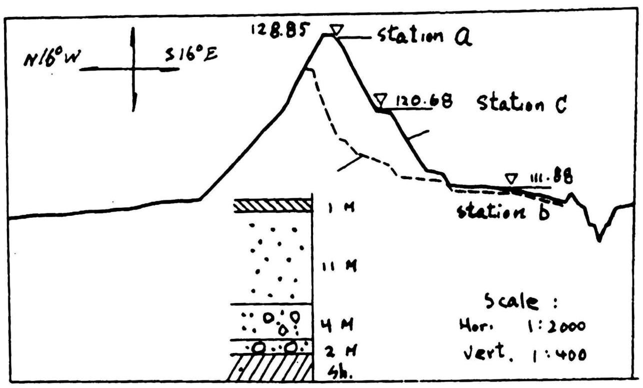
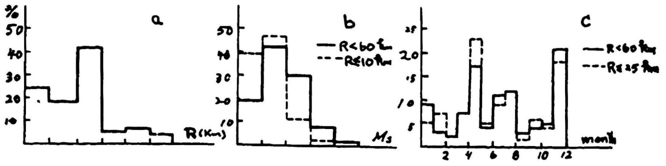


Fig.10 Sketch section showing the location of the seismic station on the auxiliary dam

* determined according to the degree of earthquake damage

In order to avoid the further unnecessary earthquake damage, soon after the Xingtai earthquake on July 1966, a seismic station was set up on the auxiliary dam at section pile number 2+677.5 for observing the seismic response of the dam (Fig.10). The corresponding data collected during 1966-1977 were drawn into figures shown in Fig.11 and Fig.12.



ig.11 Statistical distribution of the shocks of "local earthquake" according to focus distance R, magnitude Ms and month

occured. The reservoir-induced earthquakes result from the dislocation of preexisting fault and fissures by the seeping water, which is a type of stick-slip mechanism.

Though in this reservoir the induced"local earthquake" occured after the filling of reservoir were all M < 5, predominently M=2-3, they were all smaller than the design ones, but they occured very frequently and increased with time, so beside the monitoring work of the induced earthquakes, the down stream water relief work must be guaranteed for the purpose to reduce the pore water pressure.

5.2. Problem of Liquefaction

After the Xingtai earthquake, the problem of liquefaction of the saturated sand layer in the dam foundation had also been greatly con-

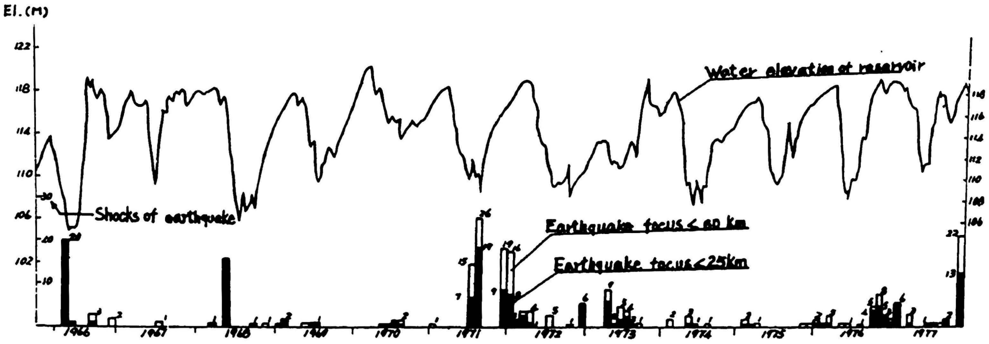


Fig.12 The relation between reservoir level and the number of shocks

om Fig.11 a and b we can find that about -50% of the "local earthquakes" are short ɔus distance and small magnitude shocks; ʒ.11-c shows the number of shocks monthly, ɔm which it can be seen in May (season of ʳ water level) and December (season of high ːer level) the chances of shocks are higher. ʒ.12 shows the process curve of the reservoir ʳel and the shocks of "local earthquake" ·ies with time. It is clear since May 1971 ʳ frequency of small earthquakes increased . occured during the period of large varian of reservoir level or at the longer duran of high water level. These are the eviden- to illustrate that the "local earthquakes" e caused by the reservoir filling. The freʼncy of "local earthquake" appeared an inasing trend in recent years. It can be erred that the "local earthquake" observed m the dam seismic station were caused by reservoir impounding on the base of favore geological condition.

ːtures and fissures of the rocks surroung and underlying the reservoir play as the ːe for water seeping. As the water seeps ɔ the deep seated fissures, increases the ɘ water pressure and decreases the friction the fissures, thus earthquake might be

cerned in dealing with the safty of the foundation against earthquake action. It can be found in Fig.4-a, a sand layer lies between the upper soil layer and the lower gravel layer, ranging from pile number 1+100 to 4+300 with thickness about 10-15 M. In order to evaluate the possibility of liquefaction of this sand layer, in 1980-1981 investigation and field test works had been carried out by drilling of two lines of bore-hole in front and behind the dam. Penetration test of the sand layer had been performed in the bore-holes. Hereby the possibility of liquefaction is studied by considering of two factors: The panetration test results and the weight of the overburden soil layer.

5.2.1. Evaluation on the possibility of liquefaction by the results of penetration test

In the "Norm of antiseismic design of hydroelectric project" of China, penetration test is required for the evaluation of liquefaction. The test is a sounding method to measure the resistance of the sand layer in the bore-hole against penetration by a specially designed device, including a dropping hammer and a split- spoon. This spoon is driven into the sand at the bottom of the bore-hole by means of the hammer (63.5 Kg) dropping dawn at a distance 76 cm from above. The number of blows needed to advance the spoon 30 cm is recorded as the penetration resistance N.

Formula (1) is used for the evaluation of liquefaction:

$$N' = \bar{N} \quad 1+0.125(d_s-3)-0.05(d_w-2) \qquad (1)$$

where,

d_s-depth of the saturated sand layer (M);

d_w-distance from the earth surface to the underground water table (M);

$\bar{N}$ -critical penetration blow of liquefaction (when d_s=3 M, d_w=2 M) being adopted as follows:

"6" for design earthquake intensity of 7^0

"10" for design earthquake intensity of 8^0

"16" for design earthquake intensity of 9^0

As the norm illustrated, if the sand layer penetration blows(N) obtained from the in-situ test in a certain depth with corresponding d_s and d_w is less than the blows calculated according to formula(1), this sand layer is to be considered as liquefied sand.

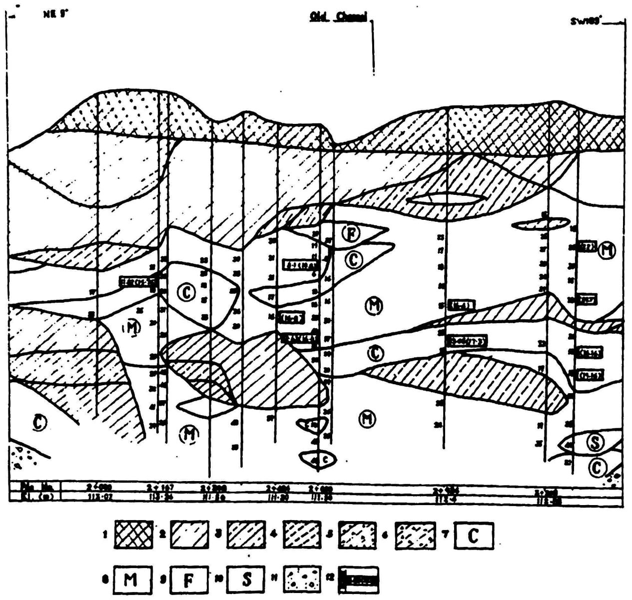


Fig.13 Geological section showing the results of penetration test

Fig.13 is a geological section in front of the dam, numerals on the left side of the bore-hole show the blows N get from in-situ penetration test and nurmals written in the rectangular frame are the blows N' calculated by formula (1) , numerals inside the small parenthesis represent blows of earthquake intensity 8^0 and numerals outside the parenthesis represent earthquake intensity 7^0. It is obvious that only a few parts of the sand layer with N less than N' which possess the possibility of liquefaction. These data demonstrates the problem of liquefaction will not be serious.

5.2.2. Evaluation on the possibility of liquefaction by considering the weight of overburden

The depth of liquefaction always depends upon the weight of overburden. During the powerful and diastrous large Tangshan earthquake(M=7.8, epicenter is about 150 Km from Beijing) on July, 28th , 1976, there happened large area of sand liquefaction in an earth dam foundation. The dam is situated only 20 Km away from the epicenter, and the earthquake intensity was estimated to be 9^0. The informations of the research on the liquefaction problem of this dam show that no liquefaction phenomena had been observed if the effective weight of overburden is larger than 1 kg/cm^2 and the thick - ness is more than 8 M.

If we compare this information with our dam foundation, the weights of the soil layer in Fig.13 are calculated and shown in Table III. It can be found that only two data of section Pile number 3+280 and 3+288 (with + symbol) are evaluated to be liquefied. This result is in agreement with the analysis of the above paragraph, which demonstrates again the problem of liquefaction would not be serious.

From our preliminary study, although the problem of liquefaction would not be serious in our dam foundation, but for a definite conclusion, further investigation and research seem to be necessary. Beside the monitoring work of the earthquake activities, the determination of wave velocity of sand layer, dynamic triaxial test of the undisturbed sand samples etc. ought to be carried out if possible.

Table III Evaluation of liquefaction by means of the weight of overburden

No.	Pile No.	Depth of the top floor of sand layer (M)	Under-ground water table(M)	Unit Weight (T/M^3)	Dry unit weight (T/M^3)	Specific gravity	Load of sand layer (Kg/Cm^2)	Evaluation of liquefaction
1	2+000	9.3	8.7	1.73	1.50	2.73	1.56	-
2	2+167	8.3	8.5	1.73	1.56	2.73	1.27	-
3	2+308	6.5	7.0	1.72	1.54	2.72	1.16	-
4	2+392	7.4	7.1	1.81	1.53	2.70	1.31	-
5	2+484	5.7	7.1	1.70	1.55	2.70	1.07	-
6	2+600	5.3	6.3	1.98	1.63	2.71	1.14	-
7	3+280	4.5	5.3	1.87	1.50	2.71	0.91	+
8	3+288	4.8	4.3	1.80	1.53	2.70	0.82	+

6. CONCLUSION

In the work of the construction of a hydropower project, excellent construction comes from good design, and good design depends upon the sufficient and correct geological informations. So in our experience, emphasis on the cooperation between geologists and engineers is of great importance. It is to say:"Our construction design must be based on the objective geological condition and analysed with the view point of Mechanics."(Tan)

REFERENCES

Tan Tjong Kie, (1982) Crucial is the correct concept. Engineering Geology and Hydrological Geology Vol.2 pp. 5-10