SEISMIC TIME HISTORY AND NON-LINEAR ANALYSIS OF LARGE-SCALE POWER HOUSE STRUCTURE

ZHANG Huidong[1], WANG Yuanfeng[2], LI Zheng[3] and WANG Rixuan[4]

[1] *Post-doctorate, School of Civil Engineering & Architecture, Beijing Jiaotong University, Beijing, China*
[2] *Professor, School of Civil Engineering & Architecture, Beijing Jiaotong University, Beijing, China*
[3] *Master, School of Civil Engineering & Architecture, Beijing Jiaotong University, Beijing, China*
[4] *Professor, School of Civil Engineering, Tianjin University, Tianjin, China*
Email: zhhuidong@yahoo.com.cn

ABSTRACT :

The previous dynamic analysis about the hydropower station structure regarded the real structure as the elastic structure. In such case, response spectrum method is usually applied on the analysis of dynamic response; In fact, cracks can be produced in the structure under a low tensile stress, since the materials of structure in practice are in the state of elastic-plasticity. In this paper, the performance of anti-seism of a large-scale hydropower station is studied with time history method, coupling with the influence of the nonlinearity of material. It can be concluded from the comparison of the linear and non-linear results that using the elastic analysis method and the elastic-plastic one can yield considerable different results. The elastic-plastic analysis method has a more distinct influence on the dynamic response of structure than that of the elastic one. It can shows the location, appearing time, and the failure behavior of the cracks with the elastic-plastic analysis method; and the time history method can depict the whole process of the variation of the dynamic response of the structure with time.

KEYWORDS: power-house; time history response; non-linear numerical analysis; anti-seism

1. INTRODUCTION

Power-houses are some special structures as its large size and complex interior structures. Anti-seismic design of the structures are not prescribed in *Code for seismic design of buildings (GB 50011-2001)*, and either not in anti-seismic design code for hydraulic structures (Qiu 1997). Therefore studies of performance-based design and damage estimation are of practical importance.

In previous studies, hydropower station structures were regarded as the structure having the property of elastic. So response spectrum method is usually applied on for the analysis of dynamic response. But when using time history method, the duration time and the peak value of ground acceleration can be considered more comprehensively and the whole process of the variation of the dynamic response of the structure under seismic can be obtained.

In the past few years, with the development of the elastic-plastic FEM theory of reinforced concrete structures, the nonlinear numerical simulation of the reinforced concrete structure has just began.

The blunt crack band model proposed by *Bazant* was extended to the case of shear-softening; and a 3-D softening constitutive relationship for the total magnitude of the yield stress was developed based on the model; in the same time, in the numerical simulation of 3-D concrete fracture analysis, a simplified secant modulus reduction coefficient was applied to linear or bilinear softening stress-strain relationship (ZHOU et al.2003). A coupling of cracking and plasticity in iterative calculation was achieved with the consideration of the cracking of concrete, in accordance with the practice of a diversion tunnel for a certain hydropower station, the whole procedure consisting of construction, operation and plugging has been analyzed (Su and Wu 2005). ANSYS Parametric Design Language was used to introduce Duncan-Chang constitutive relationship in ANSYS software platform, which is widely used in the materials such as soil and rock-fill, to implement the 3-D nonlinear FEM simulation using the middle point incremental method (Liu and Miao 2004). But in the case of mass concrete, especially the large-scale power-house structure, the studies of the 3-D nonlinear FEM simulation are rare. So it is necessary to analyze the nonlinear dynamic properties of large-scale power-house structures under seismic load. Aiming at the inadequacy of the studies about the anti-seismic of large-scale power-house structures, in this paper with an example of some large-scale power-house project, nonlinear numerical simulation of the power-house structures using time history response method is done. The results from nonlinear method are compared with that of the linear method.

2. THEORY FOR STRUCTURAL ANTI-SEISMIC ANALYSIS

2.1. Vibration Equation

In practice, ground motions are usually considered to have six degrees of freedom, which consist of three actual translational acceleration components $u_g(t)$, $v_g(t)$ and $\omega_g(t)$; and three rotational acceleration components $\theta_x(t)$, $\theta_y(t)$ and $\theta_z(t)$. However, because of the inadequacy of the data collected about rotational acceleration components, so rotational components are usually ignored in analysis. In practice, only the three translational components are considered. According to dynamic equilibrium condition, considering only three translational acceleration components, vibration equation for multi-degree-of-freedom system can be expressed as:

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=-[M]\{R\}\{\ddot{U}_g\}$$

where $\{U\}=\{\{u\}\{v\}\{\omega\}^T\}$, $\{U\}=\{\{u\}\{v\}\{\omega\}^T\}$ are vectors of relative displacement between structure and ground, and acceleration vectors of ground motion.

2.2. Solution of Vibration Equation

To solve the vibration equation, NEWMARK time integration method is used and there are some assumptions as:

$$\dot{\delta}_{t+\Delta t}=\dot{\delta}_t+\left[(1-\beta)\ddot{\delta}_t+\beta\ddot{\delta}_{t+\Delta t}\right]\Delta t$$

$$\dot{\delta}_{t+\Delta t}=\delta_t+\dot{\delta}_t\Delta t+\left[(\frac{1}{2}-\alpha)\ddot{\delta}_t+\alpha\ddot{\delta}_{t+\Delta t}\right]\Delta t^2$$

where α , β are the parameters that depend on the accuracy and stability of solutions. And then, an equation denoted by δ_t、 $\dot{\delta}_t$、 $\ddot{\delta}_t$ and $\delta_{t+\Delta t}$ can be expressed as:

$$\left(K+\frac{1}{\alpha\Delta t^2}M+\frac{\beta}{\alpha\Delta t}C\right)\delta_{t+\Delta t}=P_{t+\Delta t}+M\left[\frac{1}{\alpha\Delta t^2}\delta_t+\frac{1}{\alpha\Delta t}\dot{\delta}_t+\left(\frac{1}{2\alpha}-1\right)\ddot{\delta}_t\right]+$$

$$C\left[\frac{\beta}{\alpha\Delta t}\delta_t+\left(\frac{\beta}{\alpha}-1\right)\dot{\delta}_t+\left(\frac{\beta}{2\alpha}-1\right)\Delta t\ddot{\delta}_t\right]$$

Generally, $\alpha=0.25$, $\beta=0.5$; the vibration Equation can be transferred into a series of linear equations.

3. DYNAMIC ANALYSIS ON REINFORCED CONCRETE STRUCTURE

3.1. Failure Criterion

According to its extent of complex, the mechanics models of nonlinear seismic response of reinforced concrete structures can be divided into to several kinds such as: 2D model, truss model and finite element model. Finite element model is applied in this paper and some mechanics properties of concrete under cracking, crack closing and crushing are also considered. Because several aspects of the constitutive relationship of concrete are not sure in present, and it is also the same to the problem of the bonding properties between steel bar and concrete. Several problems involved are uncertain now. So in this paper, the bonding properties are not took into consideration.

3.2. Constitutive Relationships of Concrete and Steel Bar

The constitutive relationship of concrete under un-axial compressive stress is applied in this paper. It can be shown in Figure 1 that, in compressive area, the upward interval of the curve is second-degree parabola, and the downward interval is skew line; in the tensile area, there is second-degree parabola, hardening effects generated when tensile strain exceeds cracking strain is also taken into consideration. The ideal elastic-plastic model of steel bar is shown in Figure 2.

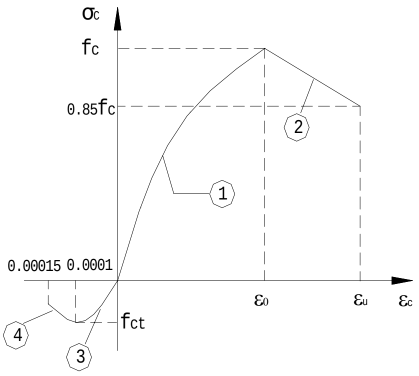

Figure 1 Constitutive relationship of concrete

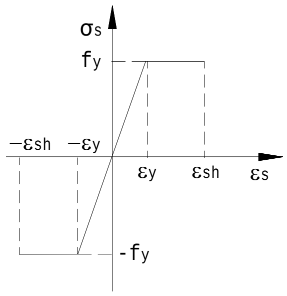

Figure 2 Constitutive relationship of steel bar

where

$$①\ \sigma_c = f_c\left[\frac{2\varepsilon}{\varepsilon_0} - \left(\frac{\varepsilon}{\varepsilon_0}\right)^2\right];\ ②\ \sigma_c = f_c\left[\frac{2\varepsilon}{\varepsilon_0} - \left(\frac{\varepsilon}{\varepsilon_0}\right)^2\right];③\ \sigma_c = \frac{2\varepsilon}{\varepsilon + 0.0001} f_{ct};④\ \sigma_c = \frac{2\varepsilon}{\varepsilon + 0.0001} f_{ct}$$

f_c, is the peak stress (prism ultimate compressive strength); ε_0 is the strain in according to the peak stress, ε_0 =0.002; ε_u is the ultimate compressive strain, ε_u =0.0033; f_{ct} is ultimate tensile strength; ε_u =0.001; ε_{sh} =0.01. $f_y = 2.1e8 N/M^2$.

3.3. Modeling of the Power-house Structure

The power-house in this paper belongs to the power house structure at the dam toe. According to the feature of construction joints between power-houses, this paper takes one units of the structure as study object in the special vibration system. The 3-D nonlinear FEM analysis is processing with dynamic interactions between the upper structures and underwater structures. The calculating model consists of spiral case, peripheral concrete of spiral case, generator pier, wind hood, dynamo floor, service gallery, machine pit entrance gallery, upper truss, horizontal stand and so on. The whole FEM model is shown in Fig.3.

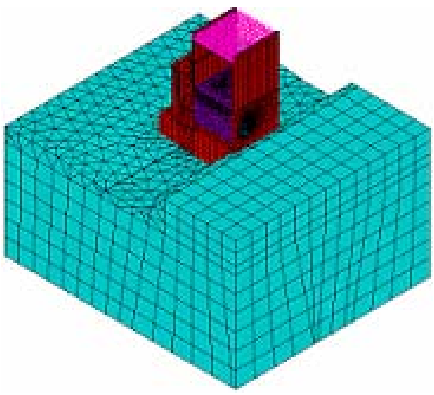

Figure 3 Powerhouse finite element model

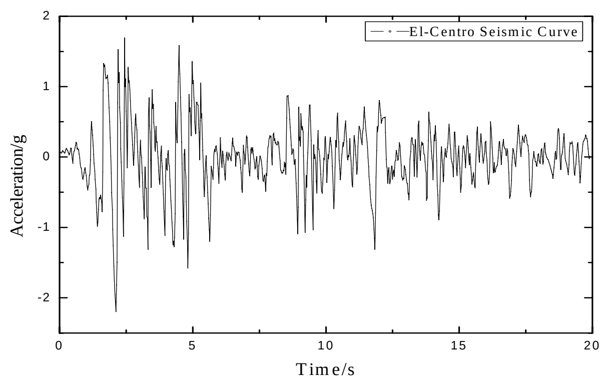

Figure 4 Seism curve of El-Centro acceleration

3.4. Material Parameters

Material properties such as elastic modulus, bulk density, and Poisson's ratio together with load information are as follows:

a、Material properties：

Material properties are given in Table3.1.

Table 3.1 Material properties

	Dynamic elastic modulus	Bulk Density	Poisson's ratio	Damping ratio
C20	3.315e4MPa	$25KN/M^3$	0.167	5%
C25	3.64e4MPa	$25KN/M^3$	0.167	5%
Bedrock	11.31e4MPa	$27KN/M^3$	0.27	5%
Truss	20.6e4MPa	$78KN/M^3$	0.3	1%

b、Roof loads：

Dead loads on top chords are $4.0Kn/m^2$; live loads on top chords are$0.7Kn/m^2$; live loads on bottom chords are $0.5Kn/m^2$; dead loads in gutter are 2.0Kn/m.

c、Definition of concrete materials

The definitions of concrete material parameters are given in Table3.2

Table 3.2 Defined data of concrete material

Shear transfer coefficients for an open crack.	0.9[5]	Ambient hydrostatic stress	-
Shear transfer coefficients for a closed crack.	0.5[5]	Biaxial crushing stress (positive) under the ambient hydrostatic stress	-
Uniaxial tensile cracking stress.	C20(C25)	Uniaxial crushing stress (positive) under the ambient hydrostatic stress	-
Uniaxial crushing stress (positive).	C20(C25)	Tensile stress reduce coefficient	0.6[5]
Biaxial crushing stress (positive).	-		

d、Simulation of concrete element

The concrete elements of large-scale power-house structures can be divided into to classes: the elements with reinforcement and the ones without reinforcement; the simulation is calculated according to the transformation of the designed reinforcement to volume reinforcement ratio.

4. NONLINEAR ANALYSIS OF TIME HISTORY RESPONSE OF POWER HOUSE STRUCTURE

4.1. Selection of Seismic Acceleration Curve

The inputs of seismic acceleration are the basis for analysis of structural seismic analysis, different seismic exciting can yield different result, especially to the structure responses under seismic load. The main selection principle is that the properties of seismic acceleration data should match the condition to the construction field. Some important parameters are as follows: seismic intensity, site classification, predominant period, response spectrum and so on. If the seismic acceleration data can not meet the selection principle, it must be adjusted to the right condition.

The power-house in this paper located on type I construction field, with fortification intensity 7 degree and the predominant period, 0.2s, according to Chinese code(*Code for seismic design of buildings(GB 50011-2001).* El-Centro seismic acceleration curve is selected and shown in Figure 4, the seismic acceleration maximum amplitude is adjusted to 0.1g, with 20s of the time duration and 0.02s of the time step length. Due to the storage capacity of the hard disc of computer, this paper inputs the ground motion into the time duration about 1.8s near the peak value of time history responses. And then the results about linear and nonlinear analysis are obtained.

4.2. Cracking Analysis

The cracks of the whole process are given in Figure 5, 6 and 7.By the results, at 0.6, a few cracks emerge within the region of the downstream wall, horizontal stand and the connection of downstream wall and floor, showing a belt like distribution; at 1.2s, when acceleration reaches to its peak value, cracks within the above two region continue to increase, and there are some cracks emerged on floors; at 1.8s, cracks continues to increase and extends to a more wider region.

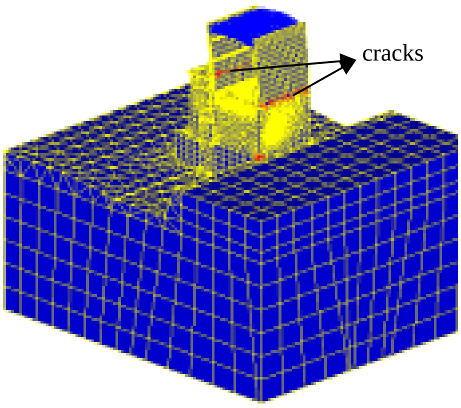

Figure 5 All cracks at 0.6s

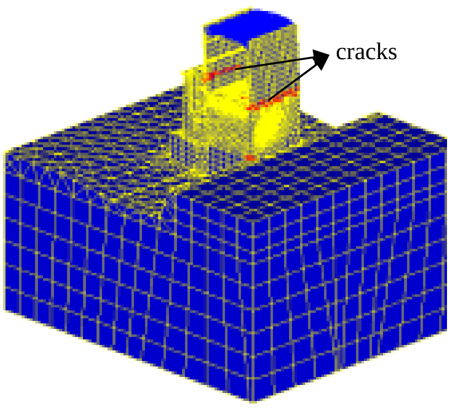

Figure 6 All cracks at 1.2s

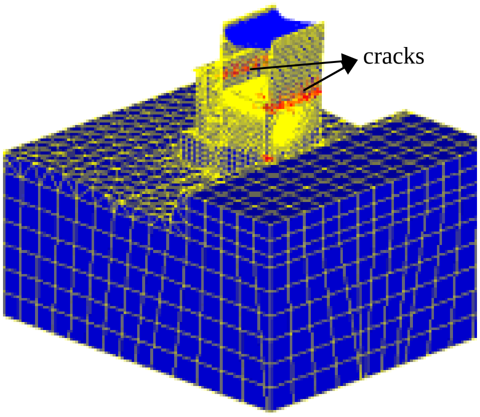

Figure 7 All cracks at 1.8s

4.3. Statistics Analysis of Nonlinear and Linear Structure Responses

The linear and nonlinear results are given by Table 3.3.

5. CONCLUSIONS

It can be concluded from the results of seismic time history and non-linear analysis of large-scale power-house structure that:

(1) Seismic time history method can have a full consideration of the peak acceleration value and its duration time. So it can depict the whole process of the variation of the dynamic response of the structure with time.

(2) According to the comparison of the principal stresses of linear and nonlinear, the whole structure can be considered to be in the elastic phase. Though the dynamic responses are small, in the point of view from the dynamic strength level, the fatigue failure of the structure under cycling loads should be given more concern.

(3) It can be shown from linear and nonlinear analysis that: structural nonlinear analysis is a more precise description of the real state of the structure under loads; and nonlinear analysis can give a more precise approximation of the interior stresses; and linear analysis can not give the time and position of the cracks emerging in the concrete.

Table 3.3 Extreme value statistics analysis of structure responses

P	C	V	Parameters of structure responses											
			Displacements(mm)			stresses(Mpa)			velocity(m/s)			acceleration (m/s/s)		
			X	Y	Z	S1	S2	S3	X	Y	Z	X	Y	Z
①	N	MAX	4.7	28.4	0.42	0.07	0.029	0	0.05	0.28	0.004	1.46	2.04	0.22
		MIN	-3.3	-38.7	-2.76	0	-0.03	-0.08	-0.08	-0.22	-0.01	-1.75	-2.63	-0.39
	L	MAX	1.6	26.5	0.26	0.03	0.028	0.002	0.012	0.25	0.002	0.7	1.65	0.24
		MIN	-1.0	-2.61	-0.17	0	-0.024	-0.03	-0.036	-0.18	-0.006	-1.23	-3.24	-0.30
②	N	MAX	3.8	9.07	0.57	1.18	0.33	0.24	0.04	0.08	0.004	1.11	0.55	0.26
		MIN	-2.6	-11.8	-4.97	-0.17	-0.39	-1.27	-0.07	-0.07	-0.01	-1.51	-1.44	-0.38
	L	MAX	1.2	6.63	0.34	0.73	0.21	0.14	0.009	0.07	0.004	0.46	0.47	0.28
		MIN	-0.7	-7.61	-0.47	-0.17	-0.24	-0.9	-0.026	-0.059	-0.009	-0.96	-1.13	-0.27
③	N	MAX	2.6	1.41	0.365	0.35	0.15	0.075	0.024	0.01	0.003	0.71	0.47	0.21
		MIN	-1.8	-0.96	-0.22	-0.06	-0.18	-0.43	-0.05	-0.03	-0.01	-1.26	-1.02	-0.34
	L	MAX	0.8	0.58	0.22	0.11	0.073	0.03	0.006	0.004	0.002	0.39	0.35	0.23
		MIN	-0.5	-0.38	-0.13	-0.02	-0.069	-0.14	-0.018	-0.012	-0.006	-0.70	-0.53	-0.26
④	N	MAX	2.0	1.13	0.41	0.95	0.14	0.019	0.019	0.010	0.004	0.5	0.33	0.17
		MIN	-1.4	-1.24	-0.3	-0.04	-0.17	-1.47	-0.03	-0.02	-0.006	-1.02	-0.9	-0.31
	L	MAX	0.7	0.54	0.3	0.98	0.007	0	0.005	0.005	0.002	0.30	0.31	0.17
		MIN	-0.4	-0.57	-0.19	0	-0.016	-1.16	-0.015	-0.008	-0.006	-0.57	-0.45	-0.27
⑤	N	MAX	2.2	1.58	0.13	0.5	0.1	0.06	0.02	0.014	0.003	0.58	0.5	0.34
		MIN	-1.45	-1.2	-0.14	-0.07	-0.11	-0.56	-0.04	-0.03	-0.004	-1.12	-1.08	-0.19
	L	MAX	0.7	0.65	0.03	0.29	0.032	0	0.005	0.005	0.001	0.35	0.33	0.23
		MIN	-0.4	-0.52	-0.03	0	-0.033	-0.34	-0.017	-0.011	-0.002	-0.61	-0.55	-0.13
⑥	N	MAX	2.33	1.49	0.09	0.38	0.04	0.01	0.02	0.013	0.0035	0.62	0.48	0.35
		MIN	-1.55	-1.03	-0.19	-0.01	-0.03	-0.32	-0.04	-0.03	-0.005	-1.18	-1.02	-0.2
	L	MAX	0.73	0.59	0.02	0.12	0.002	0	0.005	0.005	0.001	0.37	0.34	0.25
		MIN	-0.45	-0.4	-0.05	-0.001	-0.002	-0.089	-0.017	-0.012	-0.002	-0.64	-0.53	-0.13
⑦	N	MAX	1.84	1.13	0.15	0.82	0.13	0	0.017	0.01	0.0013	0.45	0.36	0.21
		MIN	-1.24	-1.22	-0.13	0	-0.17	-0.65	-0.03	-0.02	-0.003	-0.96	-0.9	-0.22
	L	MAX	0.62	0.6	0.013	0.39	0.197	0	0.004	0.006	0.001	0.27	0.30	0.18
		MIN	-0.38	-0.6	-0.01	0	-0.24	-0.46	-0.014	-0.009	-0.002	-0.53	-0.48	-0.17
⑧	N	MAX	1.48	0.89	0.18	0.45	0.05	0	0.013	0.007	0.0013	0.37	0.36	0.21
		MIN	-0.9	-0.58	-0.11	0	-0.07	-0.35	-0.03	-0.02	-0.004	-0.84	-0.74	-0.23
	L	MAX	0.49	0.34	0.01	0.12	0.01	0	0.003	0.003	0	0.255	0.31	0.19
		MIN	-0.3	-0.22	-0.07	0	-0.007	-0.11	-0.011	-0.007	-0.003	-0.451	-0.38	-0.17

Notes: ①top of downstream wall; ②middle of downstream wall; ③connection of downstream wall and floor; ④floor; ⑤beam; ⑥columns; ⑦wall of wind hood; ⑧Under-rack basis; N: nonlinear; L: linear; D: displacement; S: stress; V: velocity; A: acceleration; S1:First principal stress; S2:Second principal stress;S3:Third principal stress; MAX: maximum value; MIN: minimum value; P: position; C: category; V: value

REFERENCES

[1]Qiu, M.C.(1997).Anti-seismic design of power-house structures. *Earthquake Resistant Engineering* **2**,1-5.

[2]Zhou,Y.D.,Zhang,C.H.,and Jin,F.(2003). 3-D Nonlinear Numerical Simulation of Cracking Concrete. *Journal of Tsinghua University(Science and Technology)* **43:11**,1542-1545.

[3]Su,K., and Wu,H.G.(2005). Nonlinear finite element analysis of reinforced concrete lining of hydraulic tunnels. *Rock and Soil Mechanics* **26:9**,1485-1490.

[4]Liu,Z. and Miao, L.D.(2004). Finite element nonlinear analysis for rock-fill dam with concrete lab based on APDL. *Journal of Northwest Hydroelectric Power* **20:4**,17-20.

[5]Xia,N., and Ren, W.Q.(2006). Numerical simulation of cover layer cracking due to expansion of non-uniform corrosion of reinforcement bar and prediction of corrosion products. *Journal of Hydraulic Engineering* **37:1**, 70-74.