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3-D SEISMIC RESPONSE ANALYSIS FOR ZIPINGPU CONCRETE FACED ROCKFILL DAM

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ABSTRACT

Zipingpu concrete faced rockfill dam is located at upstream of Min River in Yangtze River drainage basin. It is the main body of Zipingpu hinge who governs irrigating, water supplying, generating electricity, preventing flood and so on. The dam is 156 meters high and 663.8 meters long. And the normal water level is 148 meters high. Investigations indicate the peak ground accelerations is 0.26g and the characteristic period of the ground is 0.17 second. Three-dimensional seismic response analysis was carried out for this dam. Improved equivalent visco-elastic model was adopted in FEM analysis. And the method of equal node force was used in residual deformation analysis. In the analysis, the foundation was supposed to be a rigid body. Time histories of the response accelerations, stresses and displacements as well as their distributions were obtained. The residual deformation induced by the input earthquake excitation was also evaluated. The analysis is of much benefit to understanding of the performance of the dam to a strong earthquake.

INTRODUCTION

In recent years, the concrete-faced rockfill (CFR) dam has been used with increasing frequency throughout the world as a result of their good performance and low cost compared with earth dams (Sherard & Cooke 1987). With its worldwide development, abundant experience in the design and construction of the CFR dams has being accumulated. As seen from the published literature, however, sufficient attention has still not been paid to the dynamic response and performance of CFR dams subjected to strong earthquake excitations. This is because the CFR dam is considered to have high fundamental safety against strong seismic shaking since (Nasim 1999): (i) the main body of the whole CFR dams is dry and hence earthquake shaking cannot cause pore water pressure and strength degradation; (ii) the reservoir water pressure acts externally on an upstream face of reinforced concrete slab and hence the entire rockfill mass acts to provide stability. It should be paid more attention that, up to now, no modern CFR dam has experienced strong earthquake shaking. In fact, most of the CFR dams that have been built are located in the areas of low seismicity, and therefore, their seismic performance has not been sufficiently considered in the design. So it is important to investigate the performance of the CFR dams under strong seismic excitations.

Presented in this paper is a brief introduction of three-dimensional earthquake response analysis for the Zipingpu CFR dam with the height of 156 meters and the length of 663.8 meters as shown in Fig.1.

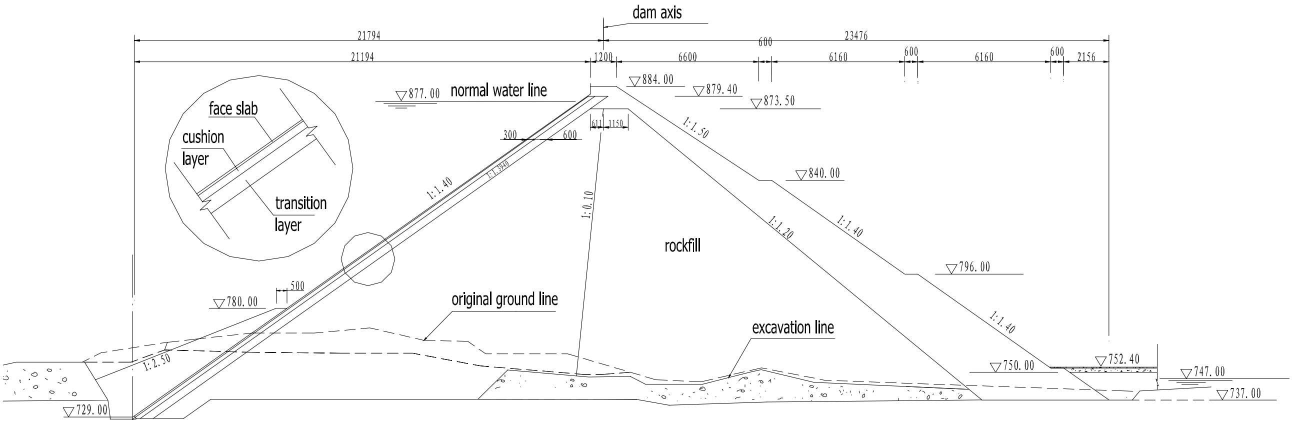


Fig.1. Typical cross section and different filled zones

ZIPINGPU CONCRETE FACED ROCKFILL DAM

The Zipingpu concrete faced rockfill dam is located at the upstream of the Min River in the Yangtze River drainage basin. It is now under construction as the main body of the Zipingpu hydraulic engineering project. This huge project was planned to act as irrigating, water supplying, generating electricity, preventing flood and so on. The designed normal water level is 148 meters high. The thickness of the face slab is variable along the height, increasing by a $0.0035h_w$ gradually from the minimum value of 0.30 m near the crest, where h_w is the depth of reservoir water (in meters). The typical cross section of the dam and the filled zones of different materials are shown in Fig.1.

MODELING AND ANALYSIS

Models and Parameters

A FEM 3D static analysis of non-linear elasto-plasticity is first made to simulate the whole construction and normal operation process of the dam. A FEM 3D seismic response analysis is then conducted to investigate the dynamic response of the dam subjected to an earthquake excitation. In this paper, only the seismic response analysis is described below.

An improved equivalent visco-elastic model (Shen 1985), which is a modification of the hyperbolic model (Hardin & Drnevich 1972), was adopted in the seismic response analysis. The dynamic stress-strain relations of soil are characterized by equivalent elastic module G and equivalent damping ratio λ. The spatial distribution of the maximum dynamic shear modulus $G_{\max}$ is estimated as a function of the effective confining pressure, $\sigma_m = (\sigma_1 + 2\sigma_3)/3$, of each rockfill element:

$$G_{max} = K \cdot p_a \left(\frac{\sigma_m}{p_a} \right)^n \tag{1}$$

in which the coefficients K and n are both functions of an effective consolidation stress ratio K_c where $K_c = \sigma_1/\sigma_3$; σ_1 and σ_3 are the maximum and minimum effective consolidation principal stresses. The data of K and n of the main fill materials determined with cyclic triaxial tests are listed in Table 1.

The relations of dynamic shear module ratio G/G_{max} and damping ratio λ versus dynamic shear strain ratio γ/γ_r are shown in Table 2. γ_r is reference shear strain, defined as

$$\gamma_r = \tau_{max} / G_{max} \tag{2}$$

$$\tau_{\max} = \left[\left(\frac{\sigma_1 + \sigma_3}{2} \sin\phi + c\cos\phi \right)^2 - \left(\frac{\sigma_1 - \sigma_3}{2} \right)^2 \right]^{0.5} \tag{3}$$

$\tau_{\max}$ is the maximum dynamic shear stress; σ_1 and σ_3 are the maximum and minimum effective consolidation principal stress respectively; φ and c are the static effective shear strength parameters.

Table 1. The Values of Coefficients K and n

Material	K_c	K	n
Rockfill	1.5	3592.3	0.430
	2.0	3784.4	0.416
	2.5	3815.6	0.424
Transition layer	1.5	2475.7	0.528
	2.0	3183.6	0.509
	2.5	3950.4	0.457
Cushion layer	1.5	2529.5	0.497
	2.0	3051.7	0.505
	2.5	3662.6	0.464

Table 2. Relations of Dynamic Shear Module Ratio and Damping Ratio versus Dynamic Shear Strain Ratio

Material	Rockfill		Transition Layer		Cushion Layer	
	K_c=2.5		K_c=2.0		K_c=2.0	
γ/γ_r	G/G_{max}	λ(%)	G/G_{max}	λ(%)	G/G_{max}	λ(%)
3×10^{-3}	1.000	0.1	1.000	0.1	1.000	0.1
10^{-2}	0.990	0.3	0.993	0.4	0.992	0.4
10^{-1}	0.653	3.1	0.785	4.2	0.740	4.6
1	0.340	9.2	0.385	10.3	0.340	10.2
2	0.245	11.2	0.320	12.1	0.268	12.2

The residual volumetric and shear strains are calculated by the following empirical formulas (Shen 1985):

$$\Delta\varepsilon_v = c_1(\gamma_d)^{c_2} \exp(-c_3 S_l^{\,2}) \frac{\Delta N}{1+N} \tag{4}$$

$$\Delta\gamma = c_4(\gamma_d)^{c_5} S_l^{\,2} \frac{\Delta N}{1+N} \tag{5}$$

and

$$N = \frac{\sum \gamma_d}{\bar{\gamma}_d} \tag{6}$$

$$S_l = \frac{(\sigma_1 - \sigma_3)(1 - \sin\varphi)}{2c\cos\varphi + 2\sigma_3 \sin\varphi} \tag{7}$$

where $\bar{\gamma}_d$ is the average dynamic shear strain amplitude for some time interval; $\sum\gamma_d$ is the sum of the averaged dynamic shear strain amplitude; N is the equivalent cyclic numbers of loading; ΔN is the increment of N for some time interval; S_l is the level of static deviator stress mobilized. The values of the parameters c_1, c_2, c_3, c_4, c_5 are shown in Table 3.

Table 3. Residual Deformation Parameters

Material	c_1	c_2	c_3	c_4	c_5
Rockfill	0.00056	0.75	0	0.040	0.75
Cushion Layer	0.00050	0.75	0	0.035	0.75
Transition Layer	0.00050	0.75	0	0.035	0.75
Concrete	0	1	1	0	1

The residual strains were calculated only at the end of each big time step (one second), and it is translated into node forces for the analysis of next step.

Calculation Condition

The 3D FEM mesh is shown in Fig.2 where there are totally 3471 nodes and 2890 elements. The static analysis is first carried out to obtain the effective mean principal stress σ_m in all the elements.

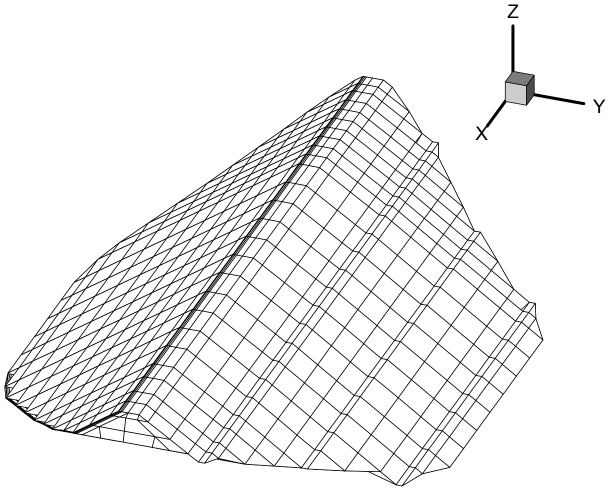


Fig.2. 3D FEM mesh

Figures 3 and 4 show the distributions of the maximum and minimum static principal stresses at the maximum cross section of the dam.

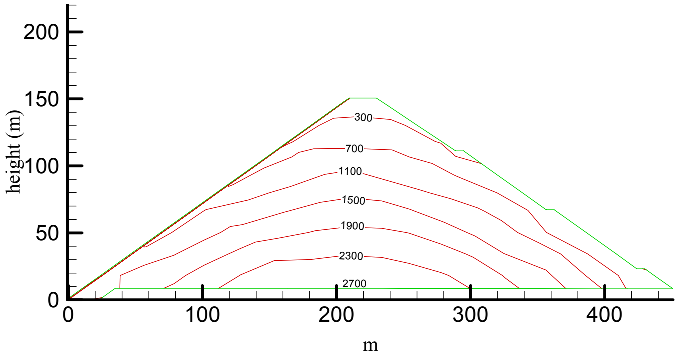


Fig.3. The contour of maximum static principal stress at the maximum cross section (in kPa)

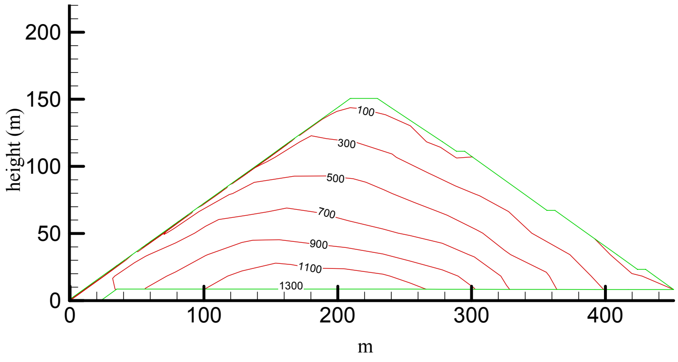


Fig.4. The contour of minimum static principal stress at the maximum cross section (in kPa)

The dynamic response analysis was made under a normal retaining water level of 148 meters high. The foundation of the dam is supposed to be a rigid body. The peak ground acceleration is 0.26g and the natural period of the ground is 0.17 second. The duration of the input earthquake is 17 seconds long. Figure 5 shows time history of the horizontal component of the input ground earthquake acceleration. In the calculation, the magnitude of the vertical component is supposed to be one-third of the horizontal component. The dynamic analysis was performed with a FE code called Tsinghua-Dyn3D, which uses Wilson- θ time integration algorithm for a direct step-by-step solution.

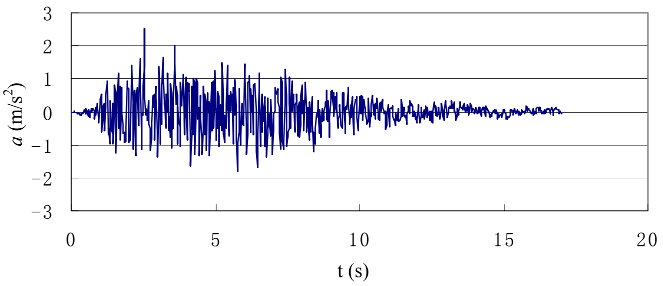


Fig.5. Time history of input earthquake acceleration

RESULTS

Acceleration Response

Time histories of the horizontal components of the response accelerations calculated for three nodes shown in Fig.6 are given in Fig.7. As seen from this figure, the peak response acceleration and the natural period of the dam increase with the increasing height; moreover, high frequency components of the shaking are significantly absorbed. The acceleration response of the dam at its crest is the strongest and the maximum response acceleration reaches about 5.3m/s^2. Similar phenomenon is also seen in Fig.8 which provides three time histories of the horizontal components of the response acceleration of the dam for three nodes of the downstream face. In addition, the earthquake response of the 3D earthquake response analysis is weaker than that of the 2D one for the three-dimensional effect.

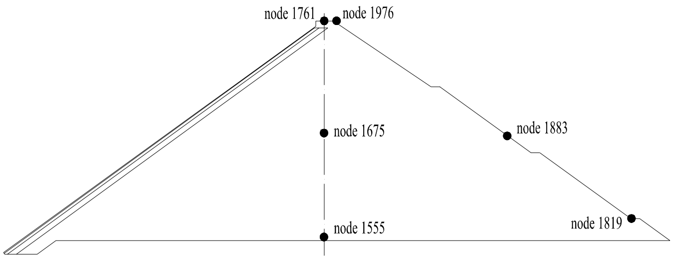
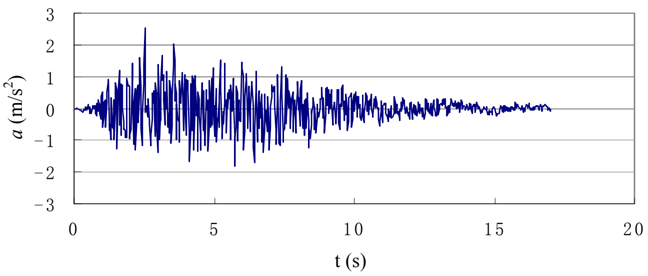
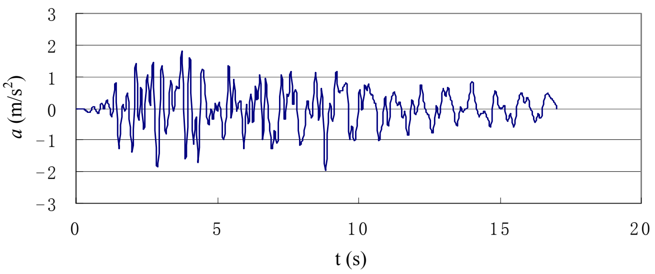


Fig.6. Positions of several typical nodes in maximum cross section

Node 1555



Node 1675



Node 1761

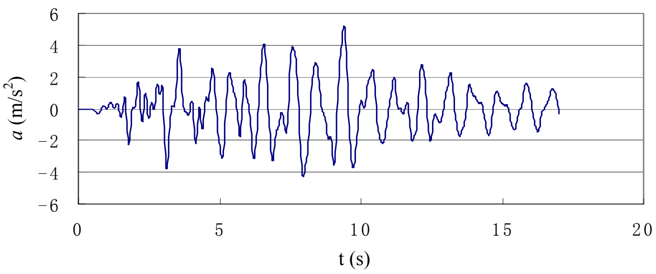
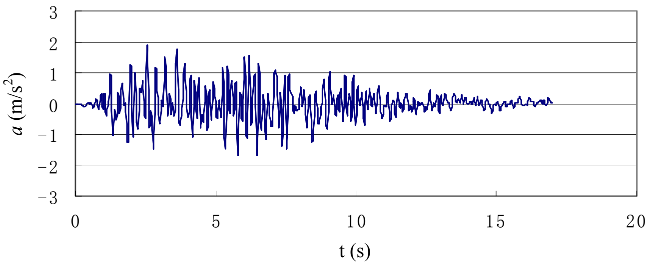
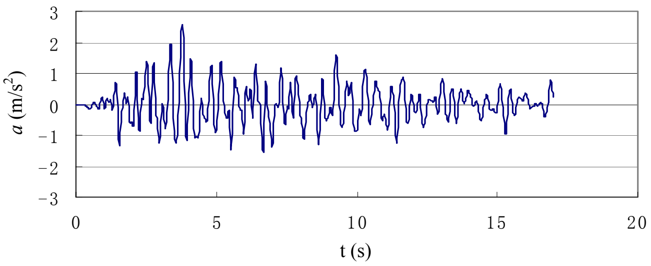


Fig.7. Time history of response acceleration of the dam at three nodes shown in Fig. 6

Node 1819 (downstream face)



Node 1883 (downstream face)



Node 1976 (downstream face)

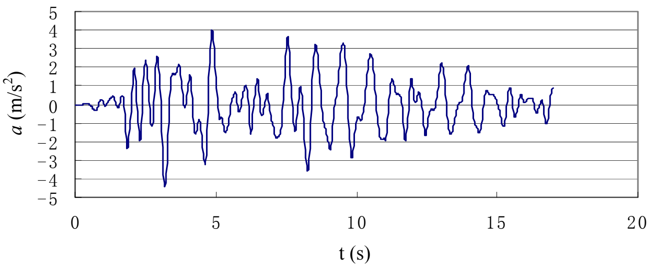


Fig.8. Time history of response acceleration of the dam at three nodes of downstream face shown in Fig. 6

Residual Deformation Distributions

Figures 9 and 10 provide the analysis results showing the distribution of the horizontal and vertical components of the earthquake-induced residual displacements in the maximum cross section. The maximum horizontal residual displacement is about 11.6cm, taking place at the middle part of the downstream slope. The maximum vertical residual displacement is about 19.3cm, occurring at the crest of the dam. From these results, the residual displacements seem not too big and their harmful effect on the dynamic stability of the dam is not significant.

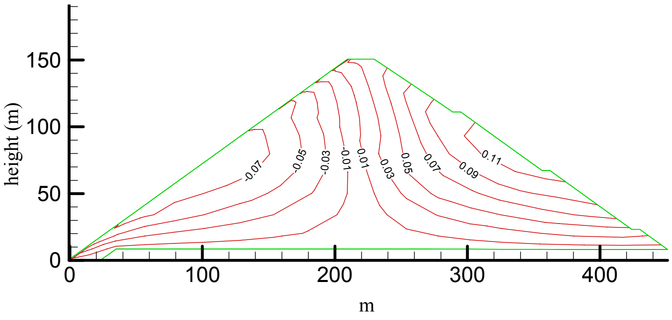


Fig.9. Distribution of horizontal residual deformation (m) at the maximum cross section

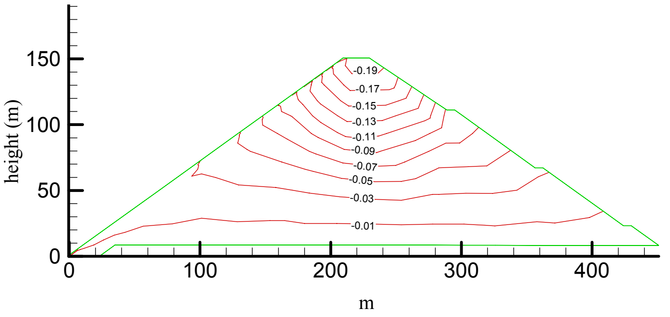


Fig.10. Distribution of vertical residual deformation (m) at the maximum cross section

Stress Distribution of Concrete Face Slab

Figure 11 shows the distribution of the initial static stress plus maximum dynamic tensile stress increment of the reinforced concrete face slab along its tangent direction. Figure 12 shows the distribution of the initial static stress plus maximum dynamic compressive stress increment of the reinforced concrete face slab along its tangent direction. It is found that the absolute values of both the tensile and compressive stresses increase significantly due to an application of earthquake shaking. The initial static stress plus maximum dynamic tensile stress increment is –2.8MPa (state of tension). The initial static stress plus maximum dynamic compressive stress increment is 10.7MPa (state of compression). Contrast to the results of the static analysis, both the values increase about twice. Obviously, the seismic action makes the tensile and compressive stresses of the face slab have a significant increase.

CONCLUSIONS

The following main conclusions may be drawn from the three-dimensional FEM seismic response analysis for the Zipingpu concrete faced rockfill dam.

1) The maximum response acceleration of 5.3m/s^2 may appear at the crest of the dam. Subjected to an application of the same earthquake excitation, the dam displays a less dynamic response for the 3D analysis than for the 2D analysis.

2) The maximum horizontal and vertical residual displacements are about 11.6cm and 19.3cm, respectively. The former may take place at the middle part of the downstream slope. The latter may appear at the crest of the dam. Both the residual displacements seem not too big and their harmful effect on the dynamic stability of the dam is not significant.

3) Under a stronger earthquake shaking with the maximum input ground acceleration of 0.26g, the absolute values of both the tensile and compressive stresses of the slab increase significantly. The former reaches its maximum value of –2.8 MPa and the latter, 10.7MPa. This may impose a harmful effect on the normal work of the slab. A further study needs to be made to understand the interface behavior of the slab and the body of dam more intensively.

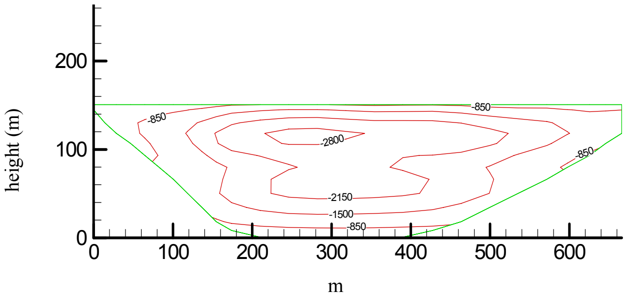


Fig.11. Distribution of initial static stress plus maximum dynamic tensile stress increment (kPa) of concrete face slab along its tangent direction

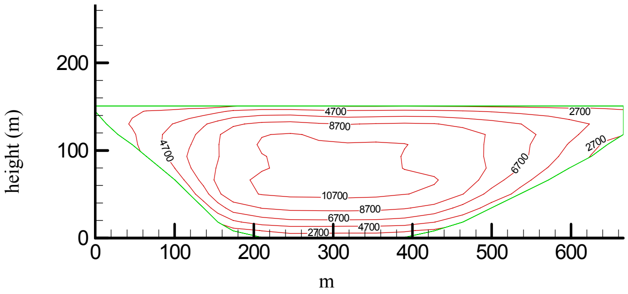


Fig.12. Distribution of the initial static stress plus maximum dynamic compressive stress increment (kPa) of concrete face slab along its tangent direction

REFERENCES

Hardin, B.O. & Drnevich, V.P. [1972]. “Shear Modulus and Damping in Soils: Design Equations and Curves”. Journal of Soil Mechanics and Foundation Division, ASCE, 98(7): 667-692.

Nasim Uddin. [1999]. “A Dynamic Analysis Procedure for Concrete-faced Rockfill Dams Subjected to Strong Seismic Excitation”. Computers and Structures 72: 409-421.

Shen Z.J. [1985]. “An Equivalent Visco-elastic Model for Liquefaction Analysis of Sands”. *Proc. 11th Int. Conf. SMFE, San Francisco:* 659-662.

Sherard, J.L. & Cooke, J.B. [1987]. Concrerte-face Rockfill Dam: I. Assessment. Journal of Geotechnical Engineering 113(10): 1096-1112.