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Impact fracture analysis of reinforced concrete rock sheds

K. Sonoda, H. Kobayashi & M. Matsuura
Department of Civil Engineering, Osaka City University, Osaka 558, Japan

ABSTRACT

Numerical analyses of impact fracture problems of reinforced concrete rock sheds are carried out using Rigid Body Spring Model (RBSM) within a frame theory. The rock sheds considered here are of a type of portal frame whose roof is covered with sand cushion. A number of numerical simulations are executed under various loading conditions such as weights of rock-mass, rock fall-heights, collisions angles of rock-fall and so on. Two types of failure mode are observed, one is bending failure mode and the other is shear failure mode due to diagonal cracks of concrete. From the numerical results obtained, the difference between the statical and the impact (dynamical) characteristics of the rock sheds are discussed.

INTRODUCTION

Main loads for design of rock sheds are rock-fall loads in addition to dead weights, earth pressure and so on. Rock-fall is an occasional phenomenon caused by relaxation of rock-slope. Since a rock-fall load is an impulsive load which incidentally occurs and is only applied during a short period less than several ten milli-seconds, it is very difficult to estimate the load in the same level as static loads such as dead weights, earth pressure and so on. A conventional design method of rock sheds in Japan is based on an allowable stress method, and rock-fall loads are placed by a static load equivalent to its dynamic peak load. But such an estimation seems to be too conservative and consequently a rational design method of rock sheds has not been realized yet.

On the other hand, a lot of computer methods to solve impact failure problems have been developed. Todays powerful softwares such as DYNA 2D and 3D may be available for design of rock sheds. Although these softwares are applicable to general impact problems, they are not economical to need too much cost for design of rock sheds.

This paper is intended to present a simplified numerical method applicable to the design of rock sheds. Rigid Body Spring Model (RBSM) originally developed by Prof. Kawai[1] is a simple model consisting of rigid bodies and springs connecting them. The model may be more effective for a static limit analysis of concrete and earth structures, but is also applicable to impact problems of these structures combining the model with an explicit time integration scheme. A contact condition between a rock-mass and sand cushion is dealt with by the Hertz theory well-known in a three dimensional elasticity. A number of numerical simulations are executed under various loading conditions such as rock-mass weights, fall-heights, collision angles of rock-fall and so on.

RIGID BODY SPRING MODEL

In RBSM used here, a portal frame of rock shed is divided into a number of rigid blocks along the frame axis as shown in Fig.1, and the material constitutive relations for concrete and reinforcements are only applied to the springs distributed over the interfaces of rigid blocks.

MATERIAL CONSTITUTIVE RELATIONS

Concrete and reinforcements

Stress-strain relation of piecewise linearization is used for concrete as shown in Fig.2. A strain softening after the peak stress in tensile region is considered, and the coefficient, α, in the strain softening region is determined considering a well-known fracture energy quantity (Cope[2]). Reinforcements are also modeled as a well-known perfectly plastic body.

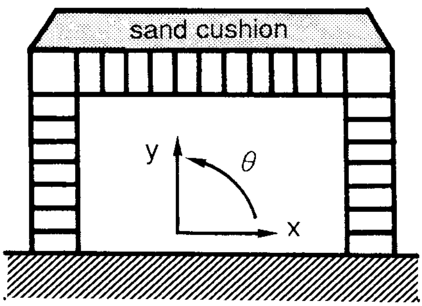

Fig.1 Element division of RBSM

Shear strength

The shear strength of concrete with shear reinforcements, V_u, is taken according to the JSCE Code prescribed for the static shear strength of RC beams, namely $V_u = V_{cu} + V_{su}$, where V_{cu} is contribution of concrete and V_{su} is contribution of reinforcements. In addition, the shear force-shear strain relation is assumed as shown in Fig. 3.

Moment and axial force calculations of RC section

To obtain the bending moment and axial force of RC section, a numerical integration over the distributed springs at the interfaces of rigid blocks is executed by dividing the whole interfacial surfaces into a number of narrow

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bands. The resulting equations are obtained as follows:

$$\left.\begin{aligned} N &= \frac{bh}{n}\sum_{i=1}^{n}\beta_i\sigma_{ci} + A_s\sigma_s + A'_s\sigma'_s \\ M &= \frac{bh}{n}\sum_{i=1}^{n}\beta_i(z_i - \frac{h}{2})\sigma_{ci} - (d' - \frac{h}{2})A_s\sigma_s - (d' - \frac{h}{2})A'_s\sigma'_s \end{aligned}\right\} \quad (1)$$

where b, h: width and depth of section; A_s, A'_s: areas of two-layer re-bars; σ_{ci}: stress of concrete; σ_s, σ'_s: stresses of re-bars; d': concrete cover; z_i: distance of band from the top; n: number of division; β_i: weight. Mechanical state of each band, which is elastic, elasto-plastic or fracture is checked at all the time steps of the explicit time integration

Moment-curvature and axial force-strain relations of RC section

Bernoulli assumption is applied to RC sections for all the states of elastic, elasto-plastic and fracture. Constitutive relations on the RC sections are derived in a form of relation between relative displacements of adjacent rigid blocks and resultant forces transmitted to their interfaces. Figures 4(a) and (b) illustrate an example of moment- or axial force-curvature relation with

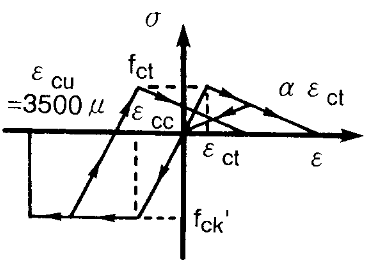


Fig.2 $\sigma-\varepsilon$ curve in concrete

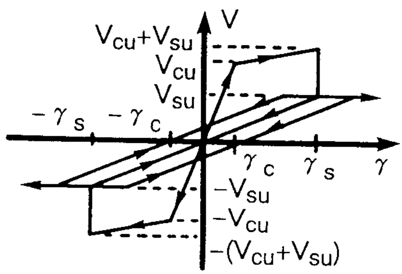
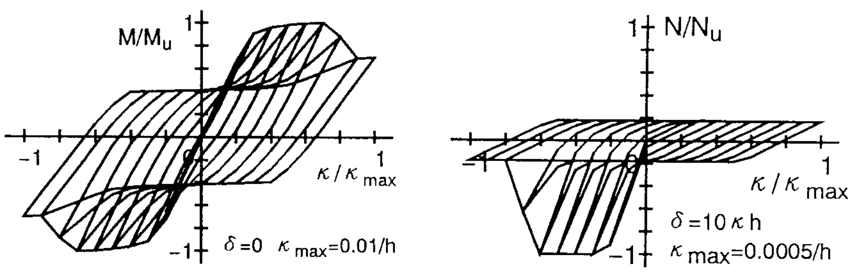


Fig.3 Shear force – strain curve



(a) M−κ relation

(b) N−κ relation

Fig.4 Hysteresis curve of RC section

Table 1: List of material properties

Reinforcement						Concrete				Sand
E_s	f_{sy}	p	p'	A_w/bS_s	S_s	E_c	f_{ct}	f_{cc}	ν_c	λ
^{a}2.1	^{b}2400	^{c}0.5	^{c}0.5	^{c}0.15	^{d}50	^{e}2.7	^{b}30	^{b}300	0.167	^{b}1000

Note: Unit ^{a} : 10^6kgf/cm^2, ^{b}:kgf/cm^2, ^{c}:%, ^{d}:cm, ^{e} : 10^5kgf/cm^2.

a controlled axial strain (δ=0 or δ= $10\kappa h$, κ: curvature). In these figures, M_u and N_u show the ultimate moment and the ultimate compressive axial force, respectively.

VERIFICATION OF RBSM

As a preliminary study, the reliability of RBSM is examined in comparison with the analytical results of a simple supported elastic beam subjected to an impulsive step load. Figure 5 shows convergency of the maximum dynamical displacement obtained by RBSM as the number of element division for the whole span length increases. The number of element division more than 20 seems to be sufficient to obtain a reliable solution.

IMPACT BEHAVIOR OF ROCK SHED BY A ROCK-FALL

Contact condition between a rock-mass and sand cushion

The Hertz theory well known in a contact problem of two elastic spheres is applied to a force-transmission condition from a rock-mass into sand cushion. As a result, a relation between impact load P(tf) and penetration D(m) of the rock-mass into sand cushion is given as

$$P = \frac{32\lambda}{9}\left(\frac{3W}{10.4\pi}\right)^{1/6} D^{3/2} \ ; \ D \leq R \qquad (2)$$

where λ: Lame's constant of sand cushion (tf/m^2); W: weight of rock-mass (tf); R: radius of rock-mass (m). An example of $P-D$ curve obtained by Eq.(2) is illustrated in Fig. 6.

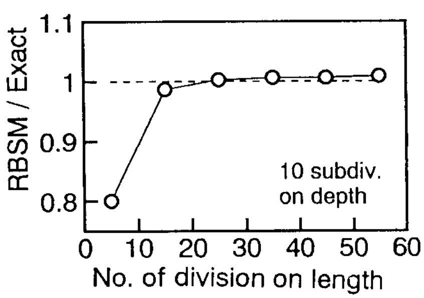


Fig.5 Convergence of solution

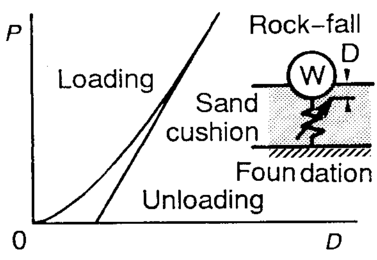


Fig.6 Contact force characteristics

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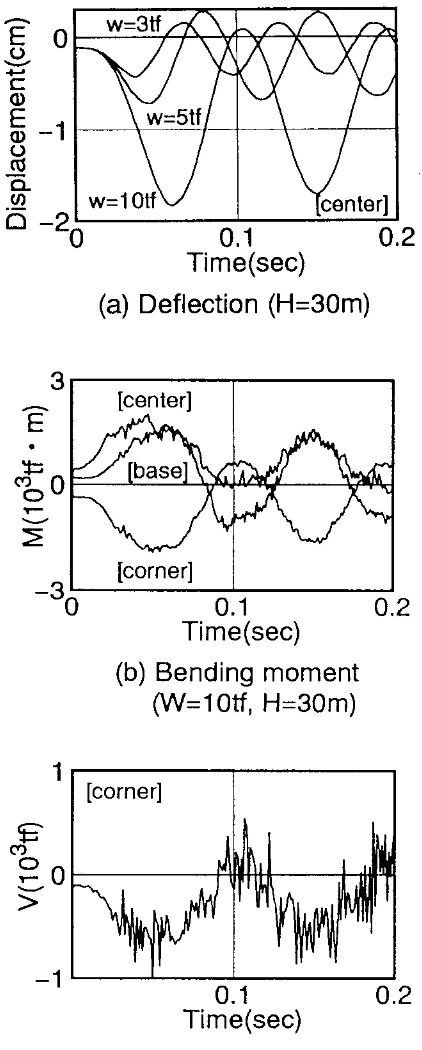
Numerical results
Geometry of the rock shed used for the numerical simulations is shown in Fig.1, in which size of the rock shed is as follows: the span length and width of roof is 10.75m and 12m respectively, the height of column is 5.6m, the depths of roof and column are 1.09m and 1.00m, respectively. The material properties are shown in Table 1.

Case of vertical fall (collision angle 90°) Figure 7(a) shows the response curve of the central deflection of roof in the cases of fall-height 30m and

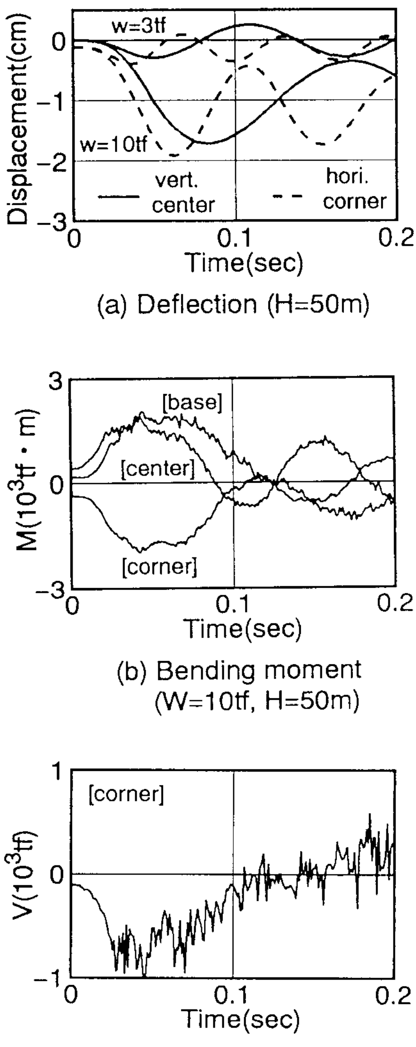


(a) Deflection (H=30m)

(b) Bending moment (W=10tf, H=30m)

(c) Shear force (W=10tf, H=30m)

Fig.7 Response in vertical fall



(a) Deflection (H=50m)

(b) Bending moment (W=10tf, H=50m)

(c) Shear force (W=10tf, H=50m)

Fig.8 Response in inclined fall

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rock-mass weights 3, 5 and 10tf. In this figure, we can observe that a failure of the roof is accelerated in the range of the rock-mass weights between 5 and 10tf, because of the growth of residual deflection in this region. In addition, we can also observe that the time at the maximum deflection is postponed as the rock-mass weight increases because of the degradation of

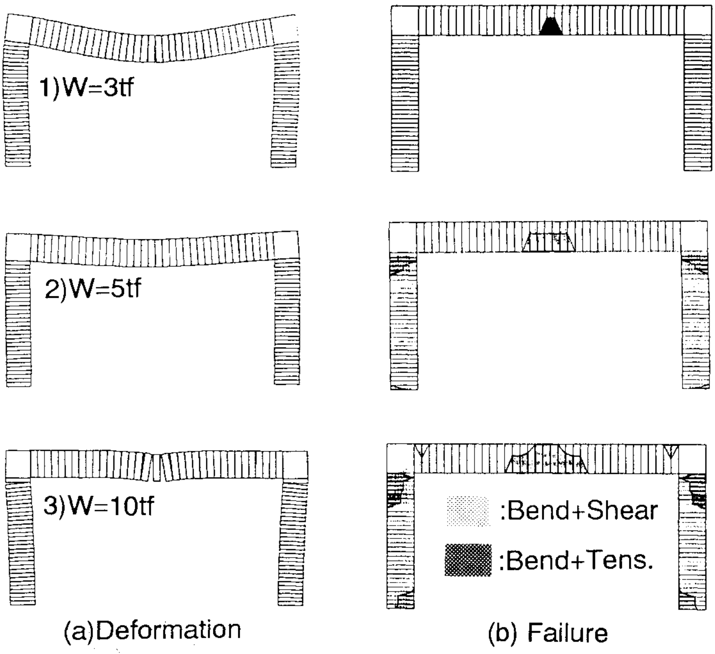

Fig.9 Deformation and material states (vertical fall : H=30m, T=0.096sec)

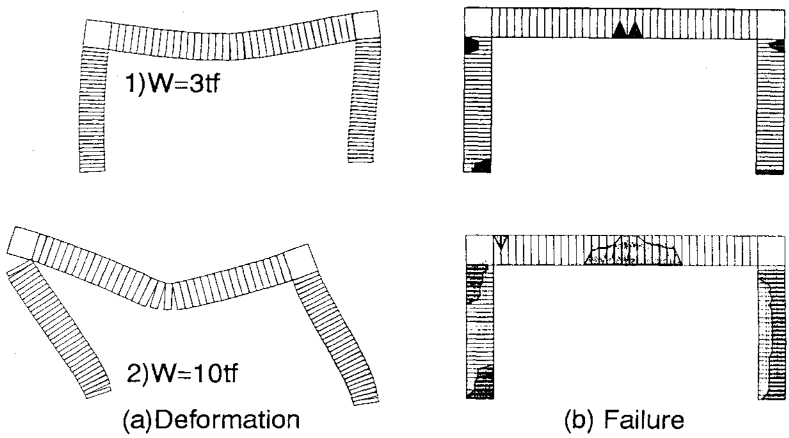

Fig.10 Deformation and material states
(inclined fall : H=50m, T=0.096sec)

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stiffness due to cracks of concrete and yielding of reinforcements. Figure 7(b) shows the responses of bending moments at the three points, namely the center and the corner of roof and the base of column. Figure 7(c) shows the shear force response at the corner of roof. It seems that these responses are mainly controlled by the first order frequency mode, but the shear force response includes more the effect of higher order frequency modes. Figures 9(a) and (b) shows variations of the deformation curves along the frame axis and the mechanical states of each element in the cases of fall-height

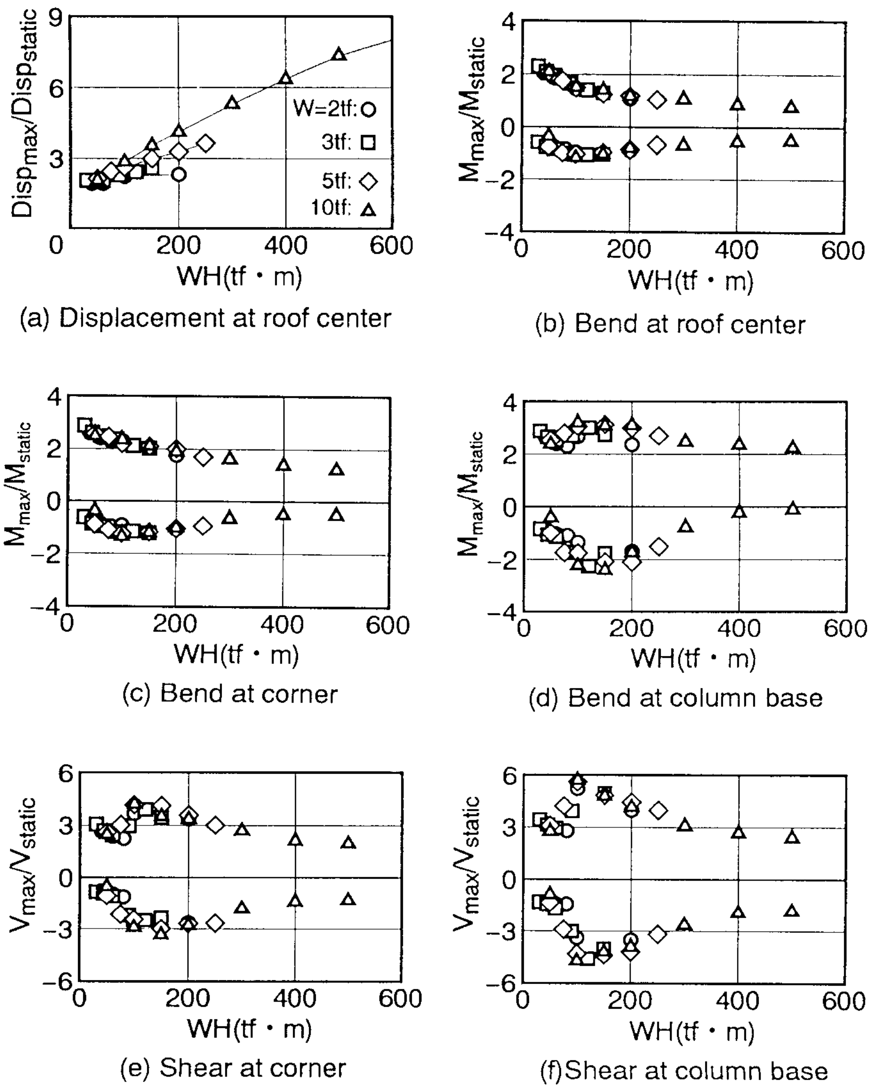

Fig.11 Dynamic magnification factors

30m. We can see that the bending failure early occurs at the section below the loading point and after then spreads to the end sections of both roof and columns. On the other hand, the shear failure following bending failure is predominant in the vicinity of loading point and finally it becomes a fatal mode.

Case of inclined fall (collision angle 45°) Figures 8(a), (b) and (c) show the responses of deflection, bending moment and shear force in the cases of the collision angle of rock-mass 45° inclined to the horizontal plane. We can observe that a horizontal failure mode becomes more dominant than a vertical failure mode in this case. Figures 10(a) and (b) show variations of the deflection curve and the mechanical states in this case.

DYNAMIC MAGNIFICATION FACTORS ON DEFLECTION, BENDING MOMENT AND SHEAR FORCE

The dynamic magnification factor is here defined by a ratio of the maximum response of impact analysis to that of elastic analysis under an equivalent static load to the dynamic peak load. Figure 11(a) shows such a factor on the central deflection of roof. It can be observed that the factor varies from 2 at the elastic stage when the potential energy of a rock-fall mass, WH, is very small to 6 or 7 at the elasto-plastic stage when WH is so large as to lead to a fatal failure. On the contrary, the factors on the bending moments at the center and the corner of roof and the base of column, as shown in Figs.11(b),(c) and (d) attain to the maximum at the elastic stage and after then gradually decreases in the elasto-plastic stage. Figures 11(e) and (f) show the factors on shear forces at the corner of roof and the base of column. These factors indicates a similar tendency to those on the bending moments, but their maximum values become larger than those.

CONCLUDING REMARKS

Rigid Body Spring Model(RBSM) have been developed to solve impact fracture problems of RC rock sheds. Since the model is based on a plane frame theory, numerical calculations using the model can very easy be done with a small computer. Therefore, the model seems to be effective for a practical use to make a rational design of RC rock sheds, if appropriate constitutive relations of materials are installed to this model. Numerical results obtained here suggested that the model has a large potentiality.

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