ABSTRACT: This paper investigates the elastic-plastic seismic behavior of long span cable-stayed steel bridges through the plane finite-element model. Both geometric and material nonlinearities are involved in the analysis. The geometric nonlinearities come from the stay cable sag effect, axial force-bending moment interaction, and large displacements. Material nonlinearity arises when the stiffening steel girder yields. The example bridge is a cable-stayed bridge with a central span length of 605 m. The seismic response analyses have been conducted from the deformed equilibrium configuration due to dead loads. Three strong earthquake records of the Great Hanshin earthquake of 1995 in Japan are used in the analysis. These earthquake records are input in the bridge longitudinal direction, vertical direction, and combined longitudinal and vertical directions. To evaluate the residual elastic-plastic seismic response, a new kind of seismic damage index called the maximum equivalent plastic strain ratio is proposed. The results show that the elastic-plastic effect tends to reduce the seismic response of long span cable-stayed steel bridges. The elastic and elastic-plastic seismic response behavior depends highly on the characteristics of input earthquake records. The earthquake record with the largest peak ground acceleration value does not necessarily induce the greatest elastic-plastic seismic damage.

INTRODUCTION

The reconstruction of bridges in Europe, primarily Germany, destroyed during World War II provided bridge engineers with the opportunity to apply new technology to an “old” concept in bridge design, the cable-stayed bridge, although the concept of supporting a bridge girder by inclined tension stays can be traced back to the seventh century (Podolny and Fleming 1972; Troitsky 1977; Podolny and Scalzi 1978). The increasing popularity of contemporary cable-stayed bridges among bridge engineers can be attributed to (1) the appealing aesthetics; (2) the full and efficient utilization of structural materials; (3) the increased stiffness over suspension bridges; (4) the efficient and fast mode of construction; and (5) the relatively small size of the bridge elements.

Over the past 40 years, rapid developments have been made on modern cable-stayed bridges. Cable-stayed bridges are now entering a new era, reaching a central span lengths ranging from 400 to 1,000 m or longer. The rapid progress is largely due to (1) the development of box-girders with orthotropic plate decks; (2) the manufacturing techniques of high-strength wires that can be used for cables; (3) the use of electronic computers in structural analysis and design; and (4) the advances in prestressed concrete structures.

It is well known that the increase in the center span length of cable-stayed bridges makes nonlinear analysis inevitable. For this special type of flexible, long span cable-supported bridge, nonlinear analysis is essential for evaluating the stresses and deformations induced not only by static loads but also by dynamic loads, such as vehicular traffic, wind, and earthquakes. When the center span length increases, a pronounced nonlinearity in the response may be expected, which will result in a considerable increase in the displacement and deformations of the bridge under strong shaking. In this case it is essential to understand the behavior from those dynamic loadings realistically.

A long span cable-stayed bridge exhibits nonlinear characteristics under any load level. These nonlinear sources may come from

- The sag effect of inclined cable stays
- The combined axial load and bending moment interaction effect of the girder and towers
- The large displacement effect
- The nonlinear stress-strain behavior of materials (material nonlinearity)

Many investigations have studied the dynamic behavior and seismic responses of this highly nonlinear structure. Some researchers disregarded all sources of nonlinearities, whereas others included one or more of these sources. The 2D geometric nonlinear seismic responses of cable-stayed bridges were studied by Fleming and Egeseli (1980), whereas other investigators (Nazmy and Abdel-Ghaffar 1990a,b; Abdel-Ghaffar and Nazmy 1991) studied the 3D geometric nonlinear seismic responses of long span cable-stayed bridges. Dumanoglu and Stevern (1989) analyzed the earthquake response of modern suspension bridges subjected to asynchronous longitudinal and lateral ground motion with the plane and linear finite-element model. Betti et al. (1993) dealt with the kinematic soil-structural interaction for long span cable-supported bridges. Jones and Spartz (1990) presented the results of a study that attempted to separate and estimate the mechanical and aerodynamic damping of a prototype bridge structure using full-scale ambient vibration and wind measurements, finite-element analysis of the bridge structure, and a wind tunnel model of the bridge deck section. Wilson and Gravelle (1991a) and Wilson and Liu (1991b) studied the dynamic behavior of cable-stayed bridges through a linear elastic 3D finite-element model. In their studies the modal behavior predicted by the finite-element model is compared to measured ambient vibrations of the full-scale cable-stayed bridge. Until now, however, the elastic-plastic seismic response analysis of long span cable-stayed bridges has seldom been done. In fact, large member stresses may be induced under strong ground motions.

Another important feature of long span bridges is the effects of the dead load. The dead load of a long span cable-stayed bridge may contribute 80–90% to total bridge loads. Dead loads are applied before the earthquake so that nonlinear seismic analysis should start from the deformed equilibrium configuration due to dead loads.

This paper is focused on the plane elastic and elastic-plastic seismic response behavior of a long span cable-stayed steel bridge with a center span length of 605 m. Both geometric

1Prof., Dept. of Civ. Engrg., Changsha Railway Univ., Changsha, 410075, Hunan Province, People’s Republic of China.
2Assoc. Prof., Dept. of Civ. Engrg., Nagoya Institute of Technology, Gokiso-Cho, Showa-ku, Nagoya 466, Japan.
Note. Discussion open until January 1, 2000. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on December 22, 1997. This paper is part of the Journal of Bridge Engineering, Vol. 4, No. 3, August, 1999. ©ASCE, ISSN 1084-0702/99/0003-0194—0203/$8.00 + $5.00 per page. Paper No. 17243.
and material nonlinearities are considered. Three strong earthquake records of the Great Hanshin earthquake of 1995 in Japan are used in the analysis. These earthquake records are input in the longitudinal direction [north-south (N-S) component], vertical direction [up-down (U-D) component], and combined longitudinal and vertical directions. The seismic response computations start from the deformed equilibrium configuration due to dead load. Geometric nonlinear effects are included. To evaluate the elastic-plastic seismic responses, a new kind of seismic damage index called the maximum equivalent plastic strain ratio is proposed.

**NONLINEAR CONSIDERATIONS**

A long span cable-stayed bridge is a nonlinear structural system in which the main girder is supported elastically at points along its length by inclined cable stays. Although the behavior of the material is linearly elastic, the overall load-displacement response may be nonlinear under normal design loads (Fleming and Egeseli 1980; Nazmy and Abdel-Ghaffar 1990b). The geometric nonlinear sources of long span cable-stayed bridges arise from the stay cable sag effect, the axial force-bending moment interaction of the superstructure, and large displacements.

A popular approach to account for the sagging of inclined cables is to consider a straight chord member with an equivalent modulus of elasticity first suggested by Ernst (1965). If the cable stress changes from $\sigma_1$ to $\sigma_2$ during a certain load increment, the secant value of equivalent elastic modulus over the load increment can then be used as follows (Nazmy and Abdel-Ghaffar 1990c):

$$ E_{eq} = \frac{E}{1 + \frac{(L_0 \gamma)(\sigma_1 + \sigma_2)}{24\sigma_1 \sigma_2}} $$

where $E_{eq} = \text{equivalent elastic modulus of inclined cables}$; $E = \text{cable material elastic modulus}$; $L_0 = \text{horizontal projected length of the cable}$; $\gamma = \text{weight per unit volume of the cable}$; and $\sigma = \text{cable tensile stress}$.

Similar to all nonlinear structural analysis problems, the nonlinear analysis of a long span cable-stayed bridge finally reduces to forming the nonlinear incremental equilibrium equations of the system and to solving these equations. The nonlinear finite-element method (NFEM), with the concept of continuum body discretization, has been a popular way to study the nonlinear behavior of long span cable-stayed bridges. Based on the characteristics of the geometric nonlinear sources described above, some NFEM formulations have been proposed. In the plane formulation presented by Fleming and Egeseli (1980) or the space formulation presented by Nazmy and Abdel-Ghaffar (1990a), for example, the sag effect of cables was accounted for by the Ernst’s equivalent elastic modulus concept. The updated structural geometry due to large displacements was included by updating each set of node coordinates, whereas the axial force-bending moment interaction effect was considered by introducing the stability beam functions based on structural stability analysis.

By using the Ernst’s equivalent elastic modulus concept, the secant stiffness matrix of an inclined cable stay is simply equal to the stiffness matrix of a truss element with length $L$ and cross-sectional area $A$. Truss elements can therefore be sufficiently used to model the inclined cables. Both bending and axial force members such as the stiffening girder and towers are suitably modeled by nonlinear beam elements.

It is accepted in practice to assume that the geometrical deformations of structural members in a long span cable-stayed bridge are characterized by large displacements and large rotations but small strains. With the developments of the finite-deformation theory and nonlinear FEM techniques with computers, researchers are more likely to use the finite-deformation theory to study the nonlinear behavior of long span cable-stayed bridges. There are several rigorous NFEM formulations available such as total Lagrangian formulation, updated Lagrangian formulation, and corotational formulation. Any of these formulations can include large displacements, large rotations, and small strains. Therefore, the geometric nonlinear sources of a long span cable-stayed bridge, such as the interaction between axial force and bending moment as well as large displacements, can be accurately considered.

A cable-stayed bridge is usually composed of three main structural parts (or bridge components), namely, inclined cable stays, superstructure, and towers. These structural parts may be made of different materials. The material nonlinear analysis of a long span cable-stayed bridge depends on the nonlinear stress-strain behavior of individual materials for the structural components. When some points (integration points) of an element exceed the yielding limit of the material, the stiffness matrix of the element should be revised to form the elastic-plastic stiffness matrix through the incremental plastic theory.

After the long span cable-stayed bridge is completed, and before the earthquake loading is applied, the bridge has sustained large dead load deformations and stresses in each member, which should be included. To consider the effect of dead load, the seismic response analysis should involve two steps: (1) The static analysis under dead loading to form the deformed equilibrium configuration; and (2) the seismic response analysis starting from the deformed configuration due to dead load.

In the seismic response analysis, the nonlinear differential equations of motion are directly integrated by the Newmark $\beta$-method. The integration constants $\alpha = 1/4$ and $\beta = 1/2$ are chosen, which correspond to an unconditionally stable scheme. In each time step the Newton-Raphson iteration technique is used to obtain the solution that satisfies both equilibrium equations and material constitutive relationships.

**MODELING OF EXAMPLE CABLE-STAYED BRIDGE**

**Description of Example Bridge**

The example bridge studied here is the Ming River long span cable-stayed steel bridge. The bridge central span length is 605 m, which is one of the longest central span cable-stayed bridge design in China. The bridge crosses the Ming River, Fuzhou, the capital of the Fujian Province and represents an important link in the highway between Fuzhou city and the new airport of Fuzhou. The bridge span arrangements are 90

![FIG. 1. Elevation View of Example Bridge](image-url)
where hysteric model of structural mild steel as shown in Fig. 4, the cycle loading used in the analysis is the common bilinear throughout. This assumption is accepted according to the numerical results of the example bridge.

In the elastic-plastic seismic response analysis, the elastic-plastic constitutive model of the steel girder under cyclic loading used in the analysis is the common bilinear hysteretic model of structural mild steel as shown in Fig. 4, where

\[ \sigma_1 = 235 \text{ MPa}; \quad \varepsilon_1^f = 0.0 \]  
\[ \sigma_2 = 300 \text{ MPa}; \quad \varepsilon_2^f = 0.0313 \]

Material Data

The three bridge components, namely, stiffening girder, stay cables, and towers, of the example bridge are composed of three different materials. The main stiffening girder is structural steel, while the towers are reinforced concrete. The stay cables are composed of high-strength wires. Material data of the example bridge are listed in Table 1. The cable data is summarized in Table 2, where the weight per unit volume of each stay cable depends on the number of wires in individual stay cables.

In the elastic-plastic seismic response analysis, the elastic-plastic behavior of the steel girder is only considered. In other words, stay cables and towers are assumed to remain elastic throughout. This assumption is accepted according to the numerical results of the example bridge.

The three bridge components, namely, stiffening girder, stay cables, and towers, of the example bridge are composed of three different materials. The main stiffening girder is structural steel, while the towers are reinforced concrete. The stay cables are composed of high-strength wires. Material data of the example bridge are listed in Table 1. The cable data is summarized in Table 2, where the weight per unit volume of each stay cable depends on the number of wires in individual stay cables.

The bridge dead loads are applied in the first analysis step. The dead loads of the example bridge are the uniformly distributed stiffening girder dead load \( q_d \), which is its own weight, and the weight of the reinforced concrete towers that can be obtained by the mass body density of the towers.

The boundary conditions are as follows: The joint between the stiffening girder and left tower is a fixed hinge, whereas another joint between the stiffening girder and right tower is a movable hinge. All side piers are movable hinge (roller) supports.

Input Ground Motions

To evaluate the effects of different strong ground motions on the seismic response behavior of long span cable-stayed bridges, three earthquake records are used in the analysis. They were all recorded during the Great Hanshin earthquake of 1995 in Japan. These are the Kobe record measured at the

![Image of Steel Boxed Girder](image1)

![Image of Concrete Tower](image2)

![Image of Bilinear Hysteretic Model](image3)

![Image of Material Table](image4)

![Image of Cable Data Table](image5)

![Image of Input Ground Motions](image6)
The second analysis step is the seismic response analysis. Three kinds of strong ground motions recorded during the Great Hanshin earthquake of 1995 in Japan are input along the bridge longitudinal direction (N-S component), vertical direction (U-D component), and combined longitudinal and vertical directions, respectively. Again, to investigate the effect of the dead load on the seismic response behavior of a long span cable-stayed bridge, the seismic response computations starting from the undeformed configuration and starting from the deformed equilibrium configuration due to dead loads are studied separately and compared. To show the effects of geometric nonlinearity and material nonlinearity, the following four cases are investigated:

1. Small deformation and linear elastic material seismic behavior
2. Geometric nonlinearity and linear elastic material seismic behavior
3. Small deformation and elastic-plastic material seismic behavior (steel girder)
4. Geometric nonlinearity and elastic-plastic material seismic behavior (steel girder)

In each seismic computation case, the following time-history seismic responses are recorded:

- Displacement time-history responses—the horizontal and vertical displacements at the span center, as well as the horizontal deformation at the top of the left tower
- Reaction time-history responses at the bottom of the left tower—the shear force, axial force, and bending moment
- Cable stress time-history response of inner cable No. C₁₈
- Stiffening girder internal force responses at the joints between the stiffening girder and the left tower—the axial force, shear force, and bending moment

Only parts of the results are listed in this paper. Table 5 gives the comparison between elastic and elastic-plastic maximum displacement responses under three earthquake records input along the combined longitudinal and vertical directions. Table 6 summarizes the comparison between elastic and elastic-plastic maximum reaction responses as well as maximum cable stress responses under three earthquake records input along the combined longitudinal and vertical directions. The comparisons between elastic and elastic-plastic seismic time-history responses under three earthquake records input along the combined longitudinal and vertical directions are shown in Figs. 6–8, respectively. The seismic time-history responses include displacement responses at the span center, bending moments at the bottom of the left tower, and C₁₈ cable stresses. All results of Tables 5 and 6 and Figs. 6–8 are obtained starting from the deformed equilibrium configuration due to dead loads.

![FIG. 5. First-Order Vibration Mode Shape (f₁ = 0.35836 Hz)](image-url)
From the results obtained in the elastic and elastic-plastic seismic response analyses of a long span cable-stayed bridge under three strong earthquake records, the following can be observed:

- The comparisons between seismic responses starting from the undeformed initial configuration and seismic responses starting from the deformed equilibrium configuration due to dead load have shown that the initial equilibrium configuration is essential to start the seismic response analysis (Ren 1997). Both linear and nonlinear seismic response analyses of long span cable-stayed bridges should start from the deformed equilibrium configuration due to dead loads.

- Regarding the equilibrium configuration under dead loads, the seismic time-history responses starting from the linearly deformed equilibrium configuration and starting from the nonlinearly deformed equilibrium configuration have shown that the difference is so small that linearly elastic analysis is acceptable when forming the deformed equilibrium configuration due to dead load.

- Starting from the deformed equilibrium configuration due to dead loads, it is found that there is only a small difference between linear and geometrically nonlinear seismic time-history analysis under strong ground motions, although the present center span length of the example cable-stayed bridge is up to 600 m. The geometric nonlinearity has little influence on the plane seismic response behavior of the example bridge. Therefore, small deformation seismic response analysis is acceptable for the seismic response analysis of long span cable-stayed bridges as was concluded by Fleming and Egeseli (1980) and Nazmy and Abdel-Ghaffar (1990b), if starting from the deformed equilibrium configuration due to dead load.

- Compared with seismic time-history response results under the longitudinal direction inputs, vertical direction inputs, and combined longitudinal and vertical inputs, it is found that the superposition principle is not suitable to the seismic response of long span cable-stayed bridges even in the linear seismic response case. Seismic responses under combined longitudinal and vertical inputs are not the sum of the individual longitudinal and vertical direction inputs due to the coupling effect that exists between longitudinal and vertical seismic responses. This is probably caused by the complicated structural dynamic characteristics of such long span cable-supported structures.

- From both elastic and elastic-plastic seismic maximum responses (Tables 5 and 6), it can be seen that the maximum horizontal displacement responses at the center of the span and at the top of the left tower under the Higashi Kobe record input are the largest, although the PGA of the Higashi Kobe record is the lowest among the three earthquake records. The largest maximum vertical displacement response at the span center is caused by the Kobe record, whereas the largest maximum $C_{12}$ cable stress response and the largest bending moment reaction response at the bottom of the left tower are caused by the Takatori record input. Therefore, both elastic and elastic-plastic seismic response behaviors of long span cable-stayed bridges are strongly dependent on the characteristics of ground motions. The earthquake record with the largest PGA value does not necessarily induce the greatest maximum response.

- Comparisons between elastic and elastic-plastic seismic responses (Tables 5 and 6 and Figs. 6–8) have shown that the elastic-plastic dynamic deformation of the stiffening steel girder tends to reduce the response because the elastic-plastic deformation of the girder absorbs part of the energy.

- Even under the combined longitudinal and vertical seismic input, the cable stress responses are all below the material strength limit so that the stay cables are still elastic for the example bridge.

### EVALUATION OF ELASTIC-PLASTIC SEISMIC RESPONSES

The ductility capacity and energy absorption capacity of a structural system are probably the most significant design features that affect the ability of a structure to withstand strong seismic ground motions. One of the most important problems in the elastic-plastic seismic responses is how to evaluate the intensity of the dynamic plastic deformation or the damage of the structure. Actually, engineers are more concerned with the residual deformation (plastic deformation) of the structure from severe earthquakes. This kind of residual plastic deformation is also used to judge the level of earthquake hazard. Engineers decide whether the structure can be repaired or not according to the residual deformation intensity of the structures.

Ductility indices or damage indices are commonly used to characterize the elastic-plastic seismic responses of structures. Many seismic damage parameters or damage indices have been proposed. Some of them may correlate with others. Very important feature involves the selection of appropriate damage parameters that are to be determined during a response inves-

### TABLE 5. Maximum Displacement (m) Responses

<table>
<thead>
<tr>
<th>Earthquake records</th>
<th>Horizontal Displacements at Center of Span</th>
<th>Vertical Displacements at top of left tower</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1)</td>
<td>(2)</td>
</tr>
<tr>
<td>Kobe</td>
<td>0.702</td>
<td>0.317</td>
</tr>
<tr>
<td>Takatori</td>
<td>(0.691)</td>
<td>(0.317)</td>
</tr>
<tr>
<td>Higashi Kobe</td>
<td>0.432</td>
<td>0.755</td>
</tr>
<tr>
<td></td>
<td>(0.320)</td>
<td>(0.806)</td>
</tr>
<tr>
<td></td>
<td>7.73</td>
<td>0.0569</td>
</tr>
<tr>
<td></td>
<td>(7.87)</td>
<td>(0.062)</td>
</tr>
</tbody>
</table>

Note: Values in brackets are corresponding elastic seismic response values.

### TABLE 6. Maximum Reaction and Cable Stress Responses

<table>
<thead>
<tr>
<th>Earthquake records</th>
<th>$Q$ (MN)</th>
<th>$N$ (MN)</th>
<th>$M$ (MN-m)</th>
<th>$C_{12}$ cable stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
</tr>
<tr>
<td>Kobe</td>
<td>0.0</td>
<td>651.5</td>
<td>6,696.2</td>
<td>10,552.0</td>
</tr>
<tr>
<td>Takatori</td>
<td>(0.0)</td>
<td>(651.4)</td>
<td>(8,780.9)</td>
<td>(12,379.0)</td>
</tr>
<tr>
<td>Higashi Kobe</td>
<td>0.0</td>
<td>586.1</td>
<td>7,897.7</td>
<td>12,108.0</td>
</tr>
<tr>
<td></td>
<td>(0.0)</td>
<td>(612.9)</td>
<td>(11,537.0)</td>
<td>(13,356.0)</td>
</tr>
<tr>
<td></td>
<td>0.0</td>
<td>534.5</td>
<td>6,485.2</td>
<td>10,567.0</td>
</tr>
<tr>
<td></td>
<td>(0.0)</td>
<td>(534.5)</td>
<td>(7,862.9)</td>
<td>(12,804.0)</td>
</tr>
</tbody>
</table>

Note: Values in brackets are corresponding elastic seismic response values.
FIG. 6. Comparison between Elastic and Elastic-Plastic Responses Under Kobe Record

(a) Vertical displacement responses at the center of span

(b) Bending moment responses at the bottom of left tower

(c) $C_{1u}$ cable stress response
FIG. 7. Comparison between Elastic and Elastic-Plastic Responses Under Takatori Record

(a) Vertical displacement responses at the center of span

(b) Bending moment responses at the bottom of left tower

(c) $C_{12}$ cable stress response
FIG. 8. Comparison between Elastic and Elastic-Plastic Responses Under Higashi Kobe Record

(a) Vertical displacement responses at the center of span

(b) Bending moment responses at the bottom of left tower

(c) $C_{ts}$ cable stress response
tigation. Overall displacement and overall energy dissipation are popularly used as seismic response damage parameters. They are both quantities that have a direct physical meaning. Various damage indices have been developed (Loh and Ho 1990). There are “global” and “local” damage indices depending on the characteristics of the damage parameters used. Most damage indices are based on a single-degree-of-freedom system or experiments in which the cycle numbers are limited. Actually, in the numerical computation of the elastic-plastic seismic responses for complicated structures, a large number of degrees of freedom and cycle numbers are used. Therefore, further research is needed to characterize the elastic-plastic seismic damage, which can be easily determined in the NFEM numerical computation of structural elastic-plastic seismic response.

Seismic damage indices should take into account both the maximum deformation and the cyclic effects. As a suitable seismic damage parameter, it should possess the cumulative characteristics during cyclic loading that can be conveniently determined. Plastic strains possess cumulative characteristics during cycle loading, and it is part of the direct output of the elastic-plastic seismic response computation with numerical methods such as the NFEM. The maximum equivalent plastic strain of the structural elements may therefore be a suitable parameter to evaluate the inelastic deformation intensity or the damage intensity of the structural elastic-plastic seismic response. Similar to the maximum displacement ductility ratio where the ductility index is the ratio of the maximum displacement to the yield displacement, based on the plastic strain of structural elements, a new kind of the seismic damage index called the maximum equivalent plastic strain ratio is proposed, which is defined as

\[
\mu_e = \frac{\varepsilon_{pl}^{e}}{\varepsilon_y}
\]

where \(\varepsilon_{max}^{e} = \) total maximum equivalent plastic strain accumulated in the structural elements. The denominator \(\varepsilon_y\) is used to normalize the damage index. It may be an appropriate strain as a material constant such as the yielding strain. In the following computation, this strain is chosen to be the plastic strain \(\varepsilon_y\) of (2) defined in the bilinear hysteretic model of the steel girder. Obviously, the maximum equivalent plastic strain ratio \(\mu_e\) is a kind of local damage index because the strain is a local parameter.

The maximum equivalent plastic strain ratio \(\mu_e\) of the example long span cable-stayed bridge under three strong ground motions, input along the combined longitudinal and vertical directions, are listed in Table 7. It can be found that

- Under the Kobe and Takatori earthquake record inputs, the steel girder element with the maximum equivalent plastic strain is close to the span center, whereas the element with the maximum equivalent plastic strain is located at the left auxiliary pier under the Higashi Kobe earthquake record.
- Under the Higashi Kobe earthquake record, maximum equivalent plastic strain ratio \(\mu_e\) is the largest, although the PGA of the Higashi Kobe record is not the largest among the three earthquake records. Under the Kobe earthquake record input with the largest PGA value, maximum equivalent plastic strain ratio \(\mu_e\) becomes the least. Therefore, the elastic-plastic seismic behavior or elastic-plastic seismic damage of long span cable-stayed bridges depends highly on the characteristics of input earthquake records and the dynamic properties of the structures. The earthquake record with the largest PGA value does not necessarily induce the greatest elastic-plastic seismic damage.

### CONCLUSIONS

Several observations can be made from the results of the preceding analysis:

1. The effect of the dead load must be considered in the seismic response analysis of long span cable-stayed bridges. Both linear and nonlinear seismic analyses should start from the deformed equilibrium configuration due to dead load.
2. The geometric nonlinearity has little influence on the seismic response behavior, even under strong earthquake record inputs. Small deformation seismic analysis is adequate as was concluded by Fleming and Egeseli (1980) or Nazmy and Abdel-Ghaffar (1990b).
3. Both elastic and elastic-plastic seismic behavior of long span cable-stayed bridges depend highly on the characteristics of earthquake records and the dynamic properties of the bridge. The earthquake record with the largest PGA value does not necessarily induce the greatest maximum responses and vice versa.
4. The elastic-plastic effects tend to reduce the seismic responses.
5. The elastic-plastic seismic damage index called the maximum equivalent plastic strain ratio proposed in this paper, which is a local value of the damage index, provides a measure of elastic-plastic seismic damage or ductility at the element level. It can be used in the evaluation of damage during elastic-plastic seismic response analyses. The index is particularly suitable when the NFEM is used in the elastic-plastic seismic analysis.
6. The elastic-plastic damage of long span cable-stayed bridges caused by strong ground motions is associated with their own structural dynamic properties and the characteristics of earthquake records. The earthquake record with the largest PGA value also does not induce the greatest elastic-plastic seismic damage.

### APPENDIX. REFERENCES

Nazmy, A. S., and Abdel-Ghaffar, A. M. (1990b).”Non-linear earth-

### TABLE 7. Maximum Plastic Strain Ductility Ratio

<table>
<thead>
<tr>
<th>Strain (1)</th>
<th>Earthquake Record</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kobe (2)</td>
</tr>
<tr>
<td>Maximum equivalent plastic strain (\varepsilon_{max}^{e})</td>
<td>4.7314 \times 10^{-2}</td>
</tr>
<tr>
<td>Ductility ratio (\mu_e)</td>
<td>0.0151</td>
</tr>
</tbody>
</table>


