ABSTRACT The dynamic analysis of cable-stayed bridges in seismically active regions is potentially important to highway facilities design. A quite general nonlinear dynamic analysis is developed for a realistic prediction of tower seismic performance of cable-stayed bridges. One of the most important decisions in carrying out proper analysis is to select a design earthquake that adequately represents the ground motion expected at certain site and in particular the motion that would drive the bridge structure to its critical response. So an extensive series of seismic response analyses using a wide range of ground motions are carried out. The ductility demands and base shear demands due to near-fault and long duration ground motions are compared. The stipulation of code standard input excitation limitations and the present calculated demands are discussed.

Keywords: Steel tower; cable-stayed bridge; seismic response, near fault ground motion.

1. Introduction

In recent decades, long span bridges such as cable-stayed bridges have gained much popularity due to their aesthetic appearance, technically innovative, structurally sound, efficient utilization of structural materials, increase of the horizontal navigation clearances and the economic trade off of span length cost of deep water foundation. As a result, the trend today for cable-stayed bridges is to use more shallow or slender stiffening girders combined with increasing span lengths. This structural synthesis provides a valuable environment for the nonlinear behavior due to material nonlinearities and geometrical nonlinearities of the relatively large deflection of the structure on the stresses and forces [1, 2]. After the January 17, 1995, Hyogoken Nanbu earthquake, the ductility design and dynamic analysis have been reconsidered in Japan [3-5]. The necessity has arisen to develop more efficient analysis procedures that can lead to a thorough understanding and a realistic prediction of the precise three-dimensional nonlinear dynamic response of bridge structural systems to improve the bridges seismic performance, to provide damage control and post-earthquake functionality. The complex and space bridge structures, such as cable-stayed bridges with long-span lengths, are relatively flexible to longitudinal, vertical, transverse and torsional vibrations. Then, the nonlinear three-dimensional vibration analysis is necessitated in the seismic resistant design of this kind of highway bridges.

Because of the unsatisfactory performance of highway bridges in the 1995 Hyogoken Nanbu earthquake [6-8], the Japanese Design Specifications of Highway Bridges were revised in 1996. Near field ground motions developed in the Hyogoken Nanbu earthquake were included in the 1996 Japanese design codes. The recent earthquake damage to bridges located within a few kilometers from a fault rupture clearly indicated the importance to consider the near-field ground motion effect. The intensity of acceleration was very high, and was characterized by single pulses with large acceleration and long predominant period. As well as the strong intensity, the directivity of the near-field ground motion is important in seismic design. The pulses with large intensity are generally different in a direction parallel or perpendicular to the fault plane, and depend on the amount and distribution of slip developed on the fault rupture. This is important in considering the bilateral directional excitation effect. Moreover, under seismic excitations that have relatively long durations, a structure undergoes several cycles during the forced vibration part of the response; therefore, its response depends more on the amount of energy dissipation particularly in the inelastic range.

The dynamic analysis of long-span bridges such as cable-stayed bridges in seismically active regions is potentially important to highway facilities design. A quite general methodology for nonlinear dynamic
analysis and a mathematical model are developed to estimate seismic response characteristics and for a realistic prediction of the seismic response of cable-stayed bridge tower. To simulate the tower earthquake response, a three-dimensional finite element model is established for the tower based on design drawings. Free vibration of three-dimensional framework structures is analyzed by using the consistent mass matrix based on the finite element approach. One of the most important decisions in carrying out proper is to select a design earthquake that adequately represents the ground motion expected at a particular site and in particular the motion that would drive the bridge structure to its critical response, resulting in the highest damage potential. So an extensive series of seismic response analyses using a wide range of peak ground accelerations; frequency contents and energy or duration for the records, vertical ground motion, near fault and long duration ground motions that are potentially important to bridge facilities design are carried out. The bridge structure is designed according to the current Japanese Specification for Highway Bridges. The ductility demands and base shear demands due to the near-fault and the long duration earthquake ground motions are compared and conclusions are drawn. The stipulation of code standard input excitation limitations and the calculated demands are discussed.

2. Nonlinear Dynamic Analysis Procedures

The governing nonlinear dynamic equation of the tower response can be derived by the principle of energy that the external work is absorbed by the work of internal, inertial and damping for any small admissible motion that satisfies compatibility and boundary conditions. By assembling the element dynamic equilibrium equation for the time \( t+\Delta t \) for all the elements, the incremental finite element equilibrium equation [9] can be obtained as:

\[
[M]([\ddot{u}])^{t+\Delta t} + [C][\dot{u}]^{t+\Delta t} + [K]([\Delta u])^{t+\Delta t} = ([F])^{t+\Delta t} - ([F])^t
\]  

(1)

where \([M],[C]\) and \([K]\) are the system mass, damping and tangent stiffness matrices at time \( t+\Delta t \), the tangent stiffness considers the material nonlinearities through bilinear elastic-plastic constitutive model incorporating a uniaxial yield criteria and kinematic strain hardening rule. \( \ddot{u}, \dot{u} \) and \( \Delta u \) are the accelerations, velocities, and incremental displacements at time \( t+\Delta t \), respectively. \( ([F])^{t+\Delta t} - ([F])^t \) is the unbalanced force vector. The dynamic equilibrium equation of motion considers both geometrical and material nonlinearities that affect the tangent stiffness and internal forces calculation.

In this study, the Newmark step-by-step integration method is used for the integration of equation of motion, since it has been experienced that the Newmark \( \beta \) method is the most suitable for nonlinear analysis; it has the lowest period elongation and has no amplitude decay or amplifications. In addition, the stability concern is not a problem with the variable ratio of time increments and natural period. The algorithm is unconditionally stable if \( \beta \geq (\gamma + 0.5)^2/4 \). In this study, the Newmark \( \beta \) of constant acceleration scheme for the solution of the differential equation of motion is considered for which \( \beta \) is equal to 0.25. The second numerical parameter \( \gamma \) of Newmark \( \beta \) method is set as \( \gamma = 0.5 \) to avoid a superfluous damping in the system. The equation of motion is solved for the incremental displacement using the Newton-Raphson iteration method; the stiffness matrix is updated at each increment to consider the geometrical and material nonlinearities and to speed the convergence rate.

3. Tower Finite Element Modeling

In the past two decades, construction of long-span cable-supported bridges has been very active in the world especially in Japan. Long-span bridges that attract engineers and offer many exciting technical problems will continue to be constructed. Since long-span bridges are flexible and low-damped, hence they are prone to vibrate under dynamic loading. Cable-stayed bridges exhibit some nonlinear behavior under either static or dynamic loads. The nonlinearities can be attributed to change in cable geometry due to sagging effects; interaction between axial forces and bending moments in the towers and the deck; and changes of bridge geometry due to its deflections. The steel tower of a three span continuous cable-stayed bridge located in Hokkaido, Japan is considered, in which the main span length is equal to 284m. Fig. 1 shows a schematic representation of bridge elevation. The towers are H-shaped steel structures as illustrated in Fig. 2. The tower consists of two steel legs and horizontal connected beam. The cross section of each leg has a hollow rectangular shaped with an interior stiffener, the cross-sectional size changes over the height of the tower. The geometrical properties of the tower legs and horizontal beam (strut) are summarized in Tables 1 and 2. Since the cable-stayed bridges are not structurally homogeneous, as a result the tower, deck and cable stays affect the structural response in a wide range of vibration modes. Moreover, the tower vibration dominates the earthquake resistant design of cable-stayed bridges, while the stiffening girder vibration dominates the wind resistant design [10]. The tower seismic response problem could be analyzed separately from the bridge structures, as long as restraint at the top of the tower provided by
the cables is accounted for properly. The steel tower is taken out of the cable-stayed bridge and modeled as three-dimensional frame structure through finite element model, Fig. 3. For the nonlinear elastoplastic dynamic analysis under strong seismic excitation, considering both geometric and material nonlinearities is vital to represent the complex behavior of cable-stayed bridges. A fiber flexural element is developed for the steel tower characterization and that element incorporates both geometric and material nonlinearities. All coupling among bending; twisting and stretching deformations for beam element is considered through the geometrical nonlinear theory. The stress-strain relationship of the beam element is modeled as bilinear stress strain constitutive relation with uniaxial yield criterion and kinematic hardening flow rule. The yield stress and the modulus of elasticity are equal to 355 MPa (SM490Y) and 200GPa, respectively; the strain hardening in the plastic region is equal to 0.01, and Poisson’s ratio is equal to 0.3.

For inelastic behavior, the spread of plasticity can be achieved through portions of the member cross section in addition to the ability to model zones of plasticity along the member length utilizing the member segments. Because the fiber model allows modeling of strain hardening, axial-flexural interaction, residual stresses and gradual spread of yielding within the cross section, it is a more realistic model than the common plastic hinge beam-column model. The fiber model, Fig. 4, used in this study is physically motivated from actual uniaxial test data of structural steel bars. Inelasticity of the fiber flexure element is accounted for by the division of the cross section into a number of fiber zones with uniaxial plasticity defining the normal stress-strain relationship for each zone, the element stress resultants are determined by integration of the fiber zone stresses over the cross section of the element. A stress update algorithm is implemented to allow accurate integration of the stress-strain constitutive law for strain increments, including full load reversals. The element stresses are updated from the last fully converged equilibrium state.

The nonlinear behavior of cable elements is idealized by using the equivalent modulus approach, in which each cable is replaced by a truss element with equivalent tangential modulus of elasticity $E_{eq}$ that is given by Ernst [11] as:

$$E_{eq} = E / \{1 + EA (wL)^2 / 12T^3 \}$$

where $E$ is the elasticity modulus, $L$ is the horizontal projected length of the cable, $w$ is the cable weight per unit length, $A$ is the cable cross sectional area and $T$ is the cable pretension force. It can be noticed that the nonlinearity of the cable stays originates with an increase in the loading followed by a decrease in the cable sag as a consequence the apparent axial stiffness of the cable increases. The inclined cable is represented by an equivalent straight cable element with relative axial deformation ($\Delta l$), the stiffness matrix of the cable element $K$ has the value equal to $E_{eq} A/\Delta l$ for $\Delta l > 0$, and the cable stiffness vanishes and no element force exist when shortening occurs, i.e. $\Delta l < 0$. This cable-stayed bridge has nine cables in each tower side. The dead load of the stiffening girder is considered to be equivalent to the vertical component of the cables pretension force and acted at their joints.

A spectral damping schemes of Rayleigh’s damping is used to form damping matrix as a linear combination of mass and stiffness matrices, which effectively captures the tower structures damping and is computationally efficient. The damping ratio corresponding to the frequencies of the fundamental in-plane and out-plane tower vibration modes is set to 2%. Using standard expressions [12], the mass ($\alpha$) and stiffness ($\beta$) proportional constants are 0.0836 sec^4 and 0.0041 sec, respectively. The damping coefficients for vibration modes can be computed.

For inelastic behavior, the spread of plasticity can be achieved through portions of the member cross section in addition to the ability to model zones of plasticity along the member length utilizing the member segments. Because the fiber model allows modeling of strain hardening, axial-flexural interaction, residual stresses and gradual spread of yielding within the cross section, it is a more realistic model than the common plastic hinge beam-column model. The fiber model, Fig. 4, used in this study is physically motivated from actual uniaxial test data of structural steel bars. Inelasticity of the fiber flexure element is accounted for by the division of the cross section into a number of fiber zones with uniaxial plasticity defining the normal stress-strain relationship for each zone, the element stress resultants are determined by integration of the fiber zone stresses over the cross section of the element. A stress update algorithm is implemented to allow accurate integration of the stress-strain constitutive law for strain increments, including full load reversals. The element stresses are updated from the last fully converged equilibrium state.

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Table 1 Cross section dimensions of tower parts

<table>
<thead>
<tr>
<th>Tower parts</th>
<th>Outer dimension (cm)</th>
<th>Stiffener dimension (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>I</td>
<td>240</td>
<td>350</td>
</tr>
<tr>
<td>II</td>
<td>240</td>
<td>350</td>
</tr>
<tr>
<td>III</td>
<td>240</td>
<td>350</td>
</tr>
<tr>
<td>IV</td>
<td>270</td>
<td>350</td>
</tr>
</tbody>
</table>

Table 2 Cross sectional properties of tower

<table>
<thead>
<tr>
<th>Cross sectional properties</th>
<th>Area (m$^2$)</th>
<th>Moment of Inertia ($I_x$) (m$^4$)</th>
<th>Torsion constant (m$^4$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.4856</td>
<td>0.4842</td>
<td>0.7952</td>
</tr>
<tr>
<td></td>
<td>0.0318</td>
<td>0.2372</td>
<td>0.4681</td>
</tr>
<tr>
<td>II</td>
<td>0.4560</td>
<td>0.4617</td>
<td>0.7567</td>
</tr>
<tr>
<td></td>
<td>0.0292</td>
<td>0.2031</td>
<td>0.4196</td>
</tr>
<tr>
<td>III</td>
<td>0.4016</td>
<td>0.4050</td>
<td>0.6856</td>
</tr>
<tr>
<td></td>
<td>0.0242</td>
<td>0.1785</td>
<td>0.3830</td>
</tr>
<tr>
<td>IV</td>
<td>0.4621</td>
<td>0.5680</td>
<td>0.7703</td>
</tr>
<tr>
<td></td>
<td>0.0337</td>
<td>0.2356</td>
<td>0.4875</td>
</tr>
</tbody>
</table>

4. Input Ground Motion

The Hyogoken Nanbu Earthquake of January 17, 1995 caused severe damage to buildings, highway bridges, railways, lifeline systems, port facilities, and so on. This event is the first instance in which engineering structures that were designed for the highest seismic forces in the world have been subjected to such destructive ground motions. Following the 1995 Hyogoken Nanbu earthquake, Japan Society of Civil Engineers issued "Proposal on Earthquake Resistance for Civil Engineering Structures". According to the proposal, two types of earthquake ground motions should be taken into account in earthquake resistant design of the structures. One is Level I motion of moderate intensity, which is likely to be experienced by the structures once or twice during their life time, and the other is Level II motion of extreme intensity rarely experienced during their life time. One of the most important decisions in carrying out proper is to select a design earthquake that adequately represents the ground motion expected at a particular site and in particular the motion that would drive the bridge structure to its critical response, resulting in the highest damage potential. A wide range of peak ground accelerations; frequency contents and energy or duration for the records, vertical ground motion, and near source ground motion are potentially important to bridge facilities design [13 -14].

A suite of recorded and simulated standard ground motion records are used for the nonlinear time history analysis: Three near-fault ground motion records [15] obtained during the 1995 Hyogoken-Nanbu earthquake (M7.2) and the 1994 Northridge Earthquake (Mw = 6.7), including three-components acceleration time histories recorded at JMA, JR Takatori and Sylmar-Converter STA. The near-field region of an earthquake is considered to be the region within several kilometers of the extension to the ground surface of the fault rupture plane. Even for moderate magnitude earthquakes, near-field ground accelerations, velocities, and displacements, can be quite high especially toward the direction of propagation of fault rupture. One or two distinctive large pulses with relatively short duration characterize near-field ground motions. Such large pulses are generated by forward rupture directivity effects in the fault-normal component, which are caused by the coherent summation of ground motions from extended fault planes. Furthermore, two long duration ground motion records [15] obtained during the Hachinohe record of the 1968 Tokachi-oki Earthquake (Mw = 8.2, M7.9), and the Ofunato record of the 1978 Miyagi-oki earthquake (Mw = 7.6, M7.4) that caused by a rupture of a portion of the plate boundary of the sub-ducting Pacific plate in the region east off northeastern Honshu. Together with the standard
ground motions \[4, 5\] level II introduced through Japan Highway Specification 1996 for different types of soil condition (Type I, II and III) are used in this study, to reflect a more realistic ground motion. Level II ground motions occur at a very short distance with a magnitude of about 7-7.2. The calculated responses for different records are compared. The characterizations of ground motions for improved analysis are shown in Figs. 5, 6 and 7, and their values are given in Table 3, where \(a_{\text{max}}\) is the maximum acceleration, \(v_{\text{max}}\) and \(d_{\text{max}}\) are the maximum velocity and displacement obtained by integrating accelerogram using trapezoidal rule. Predominant period is taken as period corresponding to maximum value of Fourier amplitude spectra. The ground displacement consists of two main long period cycles, the first cycle being the largest, and the subsequent ones decaying. These long period pulses are also distinguishable in the ground velocity history, where the amplitude of the positive pulses is larger than the amplitude of the negative pulses.

The peak ground acceleration is the parameter most often associated with severity of ground motion. However, it has generally come to be recognized that this is a poor parameter for evaluating the damage potential. Since, large recorded peak acceleration may be associated with a short duration impulse of high frequency (acceleration spike), or with a long duration impulse of low frequency (acceleration pulse). In the first case, most of the impulse is absorbed by the inertia of the structure with little deformation. However, a more moderate acceleration in the second case can result in a significant deformation of the structure. Since near fault impulse type ground motion results in a sudden burst of energy into the structure, which must be dissipated immediately and is characterized by a single un-directional large yield excursion. For the second case, it was suggested the use of maximum incremental velocity and maximum incremental displacement for characterizing the earthquake motion damage potential \[16, 17\]. Incremental velocity represents the area under an acceleration pulse and the area under the velocity pulse equals the incremental displacement. The earthquake ground motion attributes such as frequency content, duration, velocity, displacement, incremental velocity, and incremental displacement can have profound effects on the structural response than the peak ground acceleration, particularly in the inelastic range, which is significantly affected by cycling loading and its amplitude of input excitation.

(a) JMA Observatory (N-S)  (b) JR Takatori Observatory (N-S)  (c) Sylmar Converter STA (N-S)

**Fig. 5** Horizontal near fault ground motions in the 1994 Northridge and the 1995 Hyogoken-Nanbu Earthquakes
5. Natural Vibration Analysis

According to a number of full scale tests conducted for cables-stayed bridges and results of previous investigations, it is well known that the natural frequencies and natural mode shapes can be predicted with acceptable accuracy by means of linear elastic analyses that assume appropriate mass and stiffness distributions [18]. Depending on the fundamental frequencies of the steel towers of cable-stayed bridge in relation to the dominant frequency content of the seismic input motion, shifting the natural period $T$ of the tower would significantly reduce acceleration responses and tower member forces. The dynamic characteristics of the tower structure are considered on the assumption of fixed boundary conditions at its base. The natural vibration analysis is carried out for the previous described steel tower modal. The eigen values (natural periods) of the tower with the description of the mode shapes for the first eight modes and the corresponding effective modal mass and the damping coefficient obtained from the natural vibration analysis are listed in Table 4.
Table 4 Summary of tower principal vibration modes

<table>
<thead>
<tr>
<th>Mode order</th>
<th>Period (sec)</th>
<th>Effective mass as a fraction of total mass</th>
<th>Viscous damping percent</th>
<th>Mode type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.0723</td>
<td>33.195</td>
<td>2.00</td>
<td>H(_1)</td>
</tr>
<tr>
<td>2</td>
<td>0.9335</td>
<td>30.330</td>
<td>2.00</td>
<td>L(_1)</td>
</tr>
<tr>
<td>3</td>
<td>0.7726</td>
<td>0.000</td>
<td>2.18</td>
<td>T(_1)</td>
</tr>
<tr>
<td>4</td>
<td>0.5235</td>
<td>0.034</td>
<td>2.81</td>
<td>V(_1)</td>
</tr>
<tr>
<td>5</td>
<td>0.3751</td>
<td>1.735</td>
<td>3.68</td>
<td>L(_2)</td>
</tr>
<tr>
<td>6</td>
<td>0.3625</td>
<td>0.080</td>
<td>3.79</td>
<td>H(_2)</td>
</tr>
<tr>
<td>7</td>
<td>0.3296</td>
<td>0.000</td>
<td>4.12</td>
<td>T(_2)</td>
</tr>
<tr>
<td>8</td>
<td>0.1559</td>
<td>34.079</td>
<td>8.35</td>
<td>V(_2)</td>
</tr>
<tr>
<td>Sum</td>
<td>—</td>
<td>99.423</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

H: transverse vibration (in-plane), T: torsional vibration, L: longitudinal vibration (out-plane), V: vertical vibration

Depending on the relative amplitude of the mode shapes, these modes can be classified into the following three groups: the lateral dominant modes (right angle to bridge axis, H and longitudinal bridge axis, L); the torsional dominate modes, T; and the vertical dominate modes, V. The first significant mode with a relatively large mass participation is the lowest vibration mode with 2.072 sec period, which essentially involves transverse vibration of the entire tower structure (right angle to bridge axis). The second significant mode (0.9335 sec) is the longitudinal vibration mode (bridge axial direction) with effective mass of about 30%. The vertical vibration modes have period of 0.5235 sec to 0.1559 sec including modes 4 and 8. It is apparent that the contributions for the first and second modes are not only of larger values. But, there is another mode of vibration with large value of contribution factor showing a very complicated dynamic behavior. One interesting observation is that for this cable-stayed bridge tower the cumulative mass participation is about 63.5% after two modes. As a result, the tower seismic response is dominated by the first and second vibration modes, as shown in Fig. 8, moreover it is characterized by high strength and large ductility demands at low frequency content range of input excitation. On the other hand, the amplification of vertical vibration modes within the high frequency content range of the vertical excitation has slight effect on the tower seismic demands due to time lag of vertical motion to horizontal ground motions.

6. Nonlinear Dynamic Response

Nonlinear structural response analyses, including the use of time history inputs, are required to meet the goals of performance-based design. It is well known that different ground motion time histories matching the same response spectrum produce variations in the response of a structure subjected to nonlinear time history analysis. Specially, when the input time history is a near field pulse, this effect is accentuated to the point where small modifications of a near-field time history that have no significant effect on the response spectrum can have a major effect on the response of a structure subjected to nonlinear time history analysis. This demonstrates that the current standard of practice does not provide a reliable basis for providing near-field ground motion time histories that are specified solely on the basis of a design response spectrum. To identify damaging characteristics of near field ground motions, nonlinear dynamic structural analyses using the large set of ground motion time histories and potentially vulnerable bridges are developed. The nonlinear time history response adequately represents the demand for a high rate of energy absorption presented by near fault pulses. This is especially true for high ground motion levels that drive structures into the nonlinear range, invalidating the linear elastic assumption on which the elastic response spectrum in based. Nonlinear dynamic behavior of the cable-stayed bridge tower under a suite of ground motions representing near fault and long duration ground motions at both rock and soil sites is presented, and compared to the response under artificial standard magnitude 7 - 7.2 earthquake level II specified at Japanese highway bridge specification.
6.1 Acceleration and displacement responses

It is noticeable that the results of nonlinear dynamic response vary significantly with different earthquakes for the same structure. The acceleration responses for the JMA, JR Takatori and Sylmar records show higher acceleration response than Ofunato and Hachinohe records along the time history. The response of the near-fault records (JMA, JR Takatori and the Sylmar) shows more than gravity acceleration ($g$). The acceleration response in the right angle to bridge axis direction is dominated by the long period first vibration mode, while that in the bridge axis is dominated by the second natural vibration mode. Moreover, the acceleration response is significantly affected by the tower lateral stiffness in both direction (natural frequency) and the input excitation predominant period. The response of the JMA record in bridge axis direction shows more than twice gravity acceleration (2g) and higher response than that of JR Takatori and Sylmar records. It is interesting to see that the JR Takatori and the Sylmar records show similar acceleration responses along the time history with higher amplification of input excitation in the high frequency bridge axis direction as can be illustrated in Fig. 9.

The displacement response of the tower structure is calculated, the JMA, JR Takatori and the Sylmar records give much larger displacements in the right angle to bridge axis flexible direction. This effect in JR Takatori and Sylmar responses, is characterized by three cycles of large displacement amplitude related to the large pulse in the input ground motion, decaying rapidly after the peak excursions due to the large plastic deformation caused by the high intensity. The phenomenon of large ductility demand for JR Takatori and the Sylmar records could be attributed to the large acceleration pulses in the these ground motions. Unfavorable residual deformation in the tower after JR Takatori earthquake is attained. The JMA and long duration records (Ofunato and Hachinohe) show elastic behavior; as a result the response is characterized and dominated by tower free
vibration. But when the long duration records are scaled three times of the original ground motion acceleration amplitude, the response of scaled Ofunato record still has elastic behavior and the displacement response amplitude of the tower top seems to increase fairly linearly with the amplitude of the applied excitation; while the response of scaled Hachinohe record shows inelastic behavior and accumulation of inelastic deformation increases as the duration of the ground motion is increased, as a result the hysteretic characteristics affect ductility demand and residual deformation and the displacement response amplitude increase nonlinearly as can be shown in Figs. 10 and 11. From the curvature time history at tower base, it can be understood one direction yielding due to acceleration pulse in the JR Takatori record ground motion, the accumulation of inelastic deformation for three times scaled amplitude long duration Hachinohe ground motion.

6.2 Strength demands

The nonlinear time history response adequately represents the demand for a high rate of energy absorption presented by near fault pulses. This is especially true for high ground motion levels that drive structures into the nonlinear range and inelastic deformation, invalidating the linear elastic analysis. For the long duration records, the required strength level is much less than that required in near fault records. For the bridge axis direction that characterized by high strength, the flexural strength demand is significant low compared to that in the right angle to bridge axis direction. The JR Takatori presents the highest flexural and curvature ductility demands among input excitations as seen in Fig. 12.

![Fig. 10 Displacement time history at tower top (in the right angle to bridge axis)](image)

![Fig. 11 Curvature time history at tower base](image)
Form the moment curvature relation for JR Takatori record; it can clarify the near fault ground motion acceleration pulse effect on the tower structural response that tends to be characterized by progressive yielding in one direction. The asymmetric accumulation of inelastic deformation in one direction becomes increasingly important as the ground motion duration is increased. Since the hysteretic characteristics of yielding elements have a pronounced influence on the ductility demand and residual deformation, the effect of input excitation duration on the inelastic deformation accumulation can be obviously seen in the scaled Hachinohe record response as illustrated in Fig. 12.

For the standard ground motions level II response, which introduced through Japan Highway Specification 1996 for different types of soil condition (rock, stiff and soft), the shear and flexural strength demands are slight different compared to that of three component JR Takatori record, which can be attributed to the slight horizontal excitation bi-directional effect and slight coupling of tower vibration modes.

**Fig. 12** Moment & curvature relationship at tower base

**Fig. 13** Moment time history at tower base for standard ground motions
These demands are higher for soft soil condition ground motion compared to rock and stiff soil condition ground motion, while the standard ground motions significantly underestimate the axial strength demand, which may be attributed to the significant bi-directional and vertical ground motion effects on axial force response as shown in Figs. 13 and 14. The vertical reaction force response of three components input excitation is characterized by high frequency spike override long duration pulse response that may be attributed to the effect of the U-D component of the input excitation. It can be concluded, that the main effect of the vertical motion is the axial forces generation, which are uncoupled to that due to lateral forces and have a lower vibration period.

7. Conclusions

The seismic analysis and design of long-span bridges such as cable-stayed bridges in seismically active regions are potentially important to highway facilities design. The numerical analysis of a structure subjected to a strong earthquake is a challenging task, due to the highly non-linear nature of the phenomena involved in the response, both at the material and at the structural level. In this study, the effects of earthquake ground motions and structural characteristics on the response and performance of cable-stayed bridge tower are studied using nonlinear analysis tools for seismic structural analysis. A quite general nonlinear dynamic analysis methodology and a mathematical model are developed to estimate seismic response characteristics and for a realistic prediction of the seismic response of cable-stayed bridge tower. An extensive series of seismic response analyses using a wide range of peak ground accelerations; frequency contents and energy or duration for the records, vertical ground motion, near fault and long duration ground motions that are potentially important to bridge facilities design are carried out. The following conclusions can be drawn:

(1) The acceleration response is significantly affected by the tower lateral stiffness in both directions, the large ductility demand attained for JR Takatori and Sylmar records could be attributed to the large acceleration pulses in the near fault earthquake ground motions.

(2) The duration of ground motion almost has slight effect on the tower elastic response, while this effect become significant in inelastic behavior of tower structure, as a result, the ductility demand and energy dissipation through inelastic deformations nonlinearly and rapidly grow up.

(3) The near fault ground motions presents the highest shear, flexural and curvature ductility demands among the input excitations. The shear and flexural strength demands for standard ground motions are slight different compared to that of three component JR Takatori record.

(4) The seismic demands for standard ground motions are higher for soft soil condition ground motion compared to rock and stiff soil condition ground motions, while the standard ground motions significantly underestimate the axial strength demand.

References


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