Numerical simulation on seismic behavior of a bridge under multi-support excitations considering site effect

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Abstract. A series of numerical simulation were performed to investigate the earthquake response of a bridge under multi-support excitations considering site effect. Dynamic behavior of the bridge including the key position’s earthquake reaction and acceleration response had been demonstrated, respectively. Results show that the complex geological site had a great influence on bridge’s seismic response, especially on the superstructure. The research results could provide references for the seismic design and construction of bridge structure.

1. Introduction
During the earthquakes, seismic motions of each support of those large floating structures such as bridges were inevitably been different due to ground motion spatial variation effect, which were mainly caused by travelling wave effect, partially coherent effect, attenuation effect and local site effect [1,2]. Moreover, because more and more bridges have been constructed to satisfy the increasing demand of transportation in China, many of these bridges have to build in a complex geological ground. Consequently, according to the two factors mentioned above, numerical modeling and model tests have been performed by many researchers to investigate the seismic performance of bridge structures under multi-supports excitations considering site effects. Der Kiureghian first introduced spatial change of ground motions to consider the seismic response of large long-span bridges [3]. Some 3d analysis results show that the seismic response of cable-stayed bridges with non-uniform input increases, and the axial force of the main girder can even increase to 6-10 time [2]. In the seismic analysis of the west bay bridge in San Francisco and the coronado bridge in San Diego, California, the California department of transportation adopted the non-uniform ground motion input considering the site effect [4]. Wu had simulated an long-span, cable-stayed bridge under local site effect, the result shows that site effect can not be ignored [5,6]. Nevertheless, for different sites, especially some sites in mountainous areas, there will be certain discrepancy [7], and considering that the distance of the coherent model is often relatively too long, it is not necessarily applicable to conventional Bridges with small spans.

However, the above mentioned numerical analysis was mainly focused on regular uniform site or horizontal layered site. In view of this, in order to mitigate the risk of earthquake damage and better understanding on seismic behavior of bridge structure, a series of numerical simulation of a normal size bridge built on highly complex geological site were performed to investigate the seismic damage
mechanisms.

2. Research method

2.1 The kinematic equation of bridge under multi-support excitation

In the absolute coordinate system, the ground and the structure move together, and the structural degrees of freedom can be divided into two categories: the upper structure's degrees of freedom and the bearing's degrees of freedom connected to the foundation. At this time, according to the general form of the dynamic equation, the dynamic balance equation of the structure under multi-support excitation of seismic force can be written as:

\[
\begin{bmatrix}
M_{aa} & M_{ab} \\
M_{ba} & M_{bb}
\end{bmatrix}
\begin{bmatrix}
\dot{U}_a \\
\dot{U}_b
\end{bmatrix}
+ 
\begin{bmatrix}
C_{aa} & C_{ab} \\
C_{ba} & C_{bb}
\end{bmatrix}
\begin{bmatrix}
\dot{U}_a \\
\dot{U}_b
\end{bmatrix}
+ 
\begin{bmatrix}
K_{aa} & K_{ab} \\
K_{ba} & K_{bb}
\end{bmatrix}
\begin{bmatrix}
U_a \\
U_b
\end{bmatrix}
= 
\begin{bmatrix}
0 \\
P_b
\end{bmatrix}
\] (1)

where subscript a is the node at the non-supporting point of the structure, subscript b is the node at the supporting point of the structure. \( \dot{U} \), \( \ddot{U} \) and \( \dddot{U} \) is the acceleration, velocity and absolute displacement of the node. Matrix of \( M,C,K \) is the mass, damping and stiffness of the structure. \( P_b \) is the node force at the supporting point of the bridge structure.

Decompose the displacement into quasi-static and dynamic displacements:

\[
\begin{bmatrix}
U_a \\
U_b
\end{bmatrix}
= 
\begin{bmatrix}
U^s_a \\
U^d_a \\
U_b \\
0
\end{bmatrix}
\] (2)

where, \( U^s \) is the quasi-static displacement caused by the supporting point, and \( U^d \) is the dynamic displacement caused by the inertial force.

The first line of the expansion (1)

\[
\begin{bmatrix}
M_{aa} & M_{ab} \\
M_{ba} & M_{bb}
\end{bmatrix}
\begin{bmatrix}
\dot{U}_a \\
\dot{U}_b
\end{bmatrix}
+ 
\begin{bmatrix}
C_{aa} & C_{ab} \\
C_{ba} & C_{bb}
\end{bmatrix}
\begin{bmatrix}
\dot{U}_a \\
\dot{U}_b
\end{bmatrix}
+ 
\begin{bmatrix}
K_{aa} & K_{ab} \\
K_{ba} & K_{bb}
\end{bmatrix}
\begin{bmatrix}
U_a \\
U_b
\end{bmatrix}
= 
\begin{bmatrix}
0 \\
P_b
\end{bmatrix}
\] (3)

If the dynamic term is set as 0, then only the pseudo-static term is left in the equation, and the pseudo-static displacement is:

\[
\begin{bmatrix}
U^s_a \\
U_b
\end{bmatrix}
= -[K_{aa}]^{-1}[K_{ab}]U_b 
\] (4)

It is called the influence matrix. Substituting equations (2) and (4) into equation (3), and ignoring the influence of damping force on supporting points, equation (3) can be simplified as:

\[
\begin{bmatrix}
M_{aa} & M_{ab} \\
M_{ba} & M_{bb}
\end{bmatrix}
\begin{bmatrix}
\dot{U}^d_a \\
\dot{U}_b
\end{bmatrix}
+ 
\begin{bmatrix}
C_{aa} & C_{ab} \\
C_{ba} & C_{bb}
\end{bmatrix}
\begin{bmatrix}
\dot{U}^d_a \\
\dot{U}_b
\end{bmatrix}
+ 
\begin{bmatrix}
K_{aa} & K_{ab} \\
K_{ba} & K_{bb}
\end{bmatrix}
\begin{bmatrix}
U^d_a \\
U_b
\end{bmatrix}
= -([M_{aa}]T_{ab} + [M_{ab}]\dot{U}_b)
\] (5)

Equation (5) is the motion balance equation under multi-point excitation of the structure, and there is an independent acceleration at any supporting point, and then the structure reaction can be obtained.

2.2 Local site effect

In this paper, a practical engineering site is selected and a three-dimensional seismic response analysis is carried out on the engineering site by using the time-domain concentrated mass finite element method and a wave explicit time-domain finite element method combining the transmission boundary proposed by li xiaojun [8] and liaozhenpeng [9].

The explicit finite element method based on the central difference method can make use of the displacement values at the first two moments of each node and its surrounding nodes, and obtain the displacement values at any time of the node through simple addition, subtraction, multiplication and division, without the need to solve the simultaneous equations. This method not only keeps the flexibility of finite element method, but also saves computation time.

In addition to the nodes on the artificial boundary, all other calculation points adopt the time-domain recurrence formula as shown in equation (6) to give the whole process of seismic wave propagation in the calculation region:
\[
\{u_i^{p+1}\} = 2\{u_i^p\} - \{u_i^{p-1}\} - \frac{\Delta t^2}{M_i}\{F_i^p\} - \{P_i^p\}
\]  
(6)

where, \(\{u_i^p\}\) is the displacement vector of node \(i\) at time \(p\), \(\Delta t\) is the time step of calculation, and \(M_i\), \(\{F_i^p\}\) and \(\{P_i^p\}\) are respectively the mass, constitutive force vector and external force vector concentrated on node \(i\). The multi-transmission boundary MTF, proposed and gradually improved by Liao Zhenpeng \([9]\) and his collaborators, is a high-precision local artificial boundary in the time domain. Its basic principle is that the motion of the artificial boundary point at the present moment is expressed by the motion of the boundary point and its adjacent inner point at the previous moment and the previous moments. According to the assumption of apparent velocity and direction of wave propagation, the transmission formula is used repeatedly to eliminate the reflection of the wave. The node displacement vector at the \((p+1)\Delta t\) time of outgoing wave on the artificial boundary can be obtained recursively from equation (7):

\[
u_0^{p+1} = \sum_{j=1}^{N} (-1)^{j+1} C_j^N \nu_j^{p+1-j}
\]  
(7)

where, \(N\) is the transmission order, \(\nu_0^{p+1}\) is the displacement vector of the out-going wave at \(O\) point on the artificial boundary at \((p+1)\Delta t\) time, \(j\) is the unit distance of the out-going wave perpendicular to the artificial boundary, and \(\nu_j^{p}\) is the displacement vector of the out-going wave at time \(p\Delta t\) from \(O\) point to the inner distance \(jc\Delta t\) along the vertical direction of the boundary. \(\Delta t\) is the time step, \(c\) is the artificial wave velocity, \(C_j^N\) is the binomial coefficient, and is determined by the formula

\[C_j^N = \frac{N!}{(N-j)!j!}.
\]

3. Numerical model

3.1 Bridge model

The dynamic time-history analysis of this bridge is performed by ABAQUS finite element software. In this study, the bridge is a 15-hole concrete double-curved arch bridge, and all of which are C30 concrete. The bridge has a total length of 467.7 meter, the side span of 27.8 meter, the middle span of 31.7 meter, a deck width of 17 meter and a pier height of 20 meter. The model of the bridge is all solid units, and the concrete is C3D8 units. Figure 1 is the finite element model of the bridge and the cross-section of the bridge.

![Figure 1. Finite element model of bridge and cross-section of the bridge.](image)

3.2 Engineering site model

Figure 2 shows the section of a local non-uniform site. The site is close to the mountain and the bedrock surface is gradually deeper from left to right. Depending on the properties of each soil layer and the difference of the average shear wave velocity, it is divided into 6 layers, including silty soil, silty clay, and dolomite limestone. The bridge is erected on this site. The position of each bridge pier and the number of ground motion output (A1-A14) is shown in figure 2. The relevant parameters of the soil layer are shown in table 1. The volume of this model is 1000m×50m×100m.
Figure 2. Soil layer distribution and position of bridge pier.

Table 1. Physical and mechanics parameters of soils.

| Soil profiles | Unit Weight (KN·m³) | Shear wave velocity (m/s) | Poisson’s Ratio | Rayleigh damping coefficientβ |
|---------------|---------------------|---------------------------|----------------|-------------------------------|
| Top Silt      | 14.9                | 234                       | 0.40           | 0.20                          |
| Silt          | 14.0                | 211                       | 0.44           | 0.15                          |
| Silty Clay    | 14.6                | 288                       | 0.40           | 0.15                          |
| Clay          | 15.7                | 355                       | 0.40           | 0.14                          |
| Clay Breccia  | 15.9                | 487                       | 0.40           | 0.11                          |
| Rock          | 19.3                | 780                       | 0.34           | 0.10                          |

3.3 Input Ground Motion

The ground motion was confirmed according to the designed acceleration spectrum of 0.2g of the bridge. The acceleration time histories and Fourier spectra of the ground motions are shown in Figure 3. Its peak acceleration, duration and main vibration frequency is 222.0 gal, 32.00 s and 0.5 to 4 Hz, respectively.

Figure 3. The acceleration time histories and Fourier spectra of the input.

4. Results and analysis

4.1 Result of the bridge’s mode shape

As can be seen from the figure 4, the dynamic characteristics of the bridge structure have the following characteristics: (1) the critical mode of the bridge is 1-order mode shape of the bridge, which shows the horizontal deformation of the bridge pier along the z-axis. (2) The vibration of the main girder is more significant, the second and three mode shape appeared along the Z axis rotation and the fluctuation characteristics and the main girder appears. According to this high order mode shape, the main beam lateral bending, vertical bending and the lateral deformation of the bridge pier are the most important deformation characteristics.
4.2 Result of the engineering site

Assuming that the seismic wave is incident vertically upward from the lower half space, the time domain recursion is carried out when the incident wave reaches the bottom boundary of the calculation model at t=0, and 14 time-domain acceleration equations at the position of the bridge piers are calculated. According to the designed response spectrum, the time history of ground motion with a peak acceleration of 0.20g was fitted and its peak value was halved to serve as the incident seismic wave for site response analysis.

Figure 5 is a time-history comparison diagram of the input ground motion of the bedrock under the soil layer and the output ground motion acceleration at the surface bridge pier, and Figure 6 is a comparison diagram of the dynamic amplification coefficient. It can be seen that the peak acceleration of 14 surface ground motions output have a certain degree of amplification. Compared with ground motions of free bedrock, the maximum amplification is 1.11 time and the minimum amplification is 1.005 time. After the site response, the overall seismic dynamic amplification shift to the low-frequency direction. Moreover, the period of less than 0.4 seconds of this ground motions has a significant amplification effect. From the point of view of each output peak acceleration of A1 - A14, 227.5gal at A14 is the maximum value, and 204gal at A2 is the minimum value, and the dynamic amplification coefficient of each ground motion is different to some extent.

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**Figure 4.** Mode shape of bridge model.

**Figure 5.** Acceleration time history Comparison of input and output.
4.3 Result of the finite element analysis of bridge

The ground motions A1 to A14 calculated in the previous section are taken as the input ground motions at the 14 supports of the bridge respectively. Also uniform excitation had been carried out for comparison. The seismic response results under the uniform input and multi input of the bridge obtained through finite element time-history analysis are as follows:

Figure 7 shows the base stress of each pier and the displacement of the top of the pier under two ground motions. It can be seen that the seismic response of the substructure of the bridge is different under this two situations. Some of which were more severe and some were reduced. However, in general, the error limits is in 5 percent of each piers’ seismic reaction. Figure 8 shows the overall acceleration nephogram of the bridge. In general, the bridge amplification coefficient is increasing from bottom to top. The response of this bridge is more severe at both ends and in the middle; Figure 9 shows the acceleration response of the bridge’s deck. It can be observed that the multi-excitations strongly affect the seismic reaction of the supper structure of the bridge than the uniform excitations, the max acceleration response increase about 20 percent than the uniform excitations.

Figure 6. Dynamic amplification Comparison of input and output.
5. Conclusions

This research simulated the seismic reaction of a bridge located in complex geological site through 3-dimension finite element analysis in order to find out the seismic damage mechanism of the bridge. The following concluding remarks and recommendations could be drawn:

(1) Site effect, especial of inhomogeneity engineering site, have a great influence on the propagation of ground motion. It not only influences the peak acceleration of ground motion but also change its characteristics of the Fourier spectrum within a few hundred meters.

(2) Compared with uniform load, non-uniform load greatly affect seismic response the key parts of the bridge. Especially, for superstructure of the bridge, the max acceleration increased about 20%. This result maybe due to non-uniform excitation caused a greater interaction between bridge piers. Therefore, local site effect should be considered in structural simulation.

(3) The seismic response of bridges with different sites and structures varies greatly. In this paper, only one kind of non-uniform seismic input response of a bridge is considered. If considering many different spans and lengths, especially the long-span bridge, and then through analysis and comparison, it may be best to find the influence rule of non-uniform excitation. In addition, the spectral characteristics of the input ground motion will also have an impact on the research results presented in this paper, which should be paid attention to in future research.

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