Strain rate effect on dynamic response of bridge piers in high-speed railway under the earthquake loading

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Abstract. Both concrete and steel are sensitive to strain rate, and the high strain rate caused by earthquakes will improve the material property including the strength and elasticity modulus. While analysing the dynamic response of bridges under the earthquake loading, there are some certain errors based on static constitutive model. By the method of dynamic increasing factors (DIF) of concrete and steel, the dynamic responses of bridges in high-speed railway were analysed in fibre beam-column element models to study the strain rate effect, and the reasonable uniaxial constitutive model of concrete and steel were used in the numerical simulation. The results show that the maximum moment at the bottom of pier calculated by quasi-static constitutive models is less than that calculated by constitutive models considering strain rate. Compared with the dynamic constitutive models considering strain rate, the larger curvature at the bottom of the pier and displacement at the top of pier were obtained in the analysis on the basis of quasi-static constitutive models. The strain rate has less effect on the dynamic response before the piers yield, and the effect can be ignored. While the earthquake loading can cause piers to fail or collapse, the strain rate effect needs to be precisely considered.

1. Introduction
China's high-speed railways are widely distributed. Currently, more than 40 high-speed railway passenger dedicated lines have been built, with a total mileage of more than 35,000 kilometers [1]. The proportion of bridges in high-speed railways is generally large. Unlike highway piers and building structures, high-speed railway piers meet the requirements of better train stability and comfort through larger section stiffness, with larger cross-section dimensions and less reinforcement ratio. The author has performed many low cyclic loading tests on this type of bridge piers [2], but using static material properties to calculate the dynamic response of bridge piers under earthquakes will cause some errors [3].

The strain rate of the load on the structure caused by creep, static, earthquake, shock and explosion load gradually increases [4]. Studies show that the strain rate of static load is generally $1 \times 10^{-6} \sim 1 \times 10^{-5} \text{s}^{-1}$, the strain rate of structures under earthquakes is generally between $1 \times 10^{-5}$ and $1 \times 10^{-3} \text{s}^{-1}$ [5] [6]. Considering the strain rate effect of materials is a key issue in the transition of hysteretic response of structures under static loads to the dynamic response of structures under earthquakes.
Earlier, the characteristics of concrete materials at high strain rates were obtained by the method of the SHPB (the Slip Hopkinson Press Bar) test [7]. Subsequently, electro-hydraulic servos and other equipment were widely used in structural dynamic tests [8] [9]. In addition, drop hammer tests, light gas gun tests, and acoustic emission tests were also often used to study dynamic characteristics of concrete. Bischoff and Perry summarized the experimental data of compressive and tensile properties of concrete under different strain rates, and found that the compressive strength and elastic modulus of concrete under high strain rates have an increasing trend, with strain rates higher than 1s⁻¹, the dynamic increase factor of concrete material properties is significantly increased [10]. Rossi and Toutlemonde explained that this phenomenon may be caused by the effect of inertial force [11]. Malvar and Ross also summarized the experimental data of the dynamic tensile properties of concrete under different strain rates. Compared with the dynamic compression test, the tensile test has fewer data and the test results are controversial, but it is generally believed that the tensile strength increases with increasing strain rate [12].

It is found that the load rate, the material's rate sensitivity, the crack development mode, and the inertial force all affect the ductility and failure mode of the structure. The ductility of the structure decreases at high strain rates, and the failure mode shifts to shear failure. Many uniaxial constitutive models of reinforced and concrete materials considering strain rate effects have been proposed, of which the dynamic increment factor (DIF) method is the most commonly used. This paper introduces the widely used empirical formula for the DIF of materials and applies it to the uniaxial material constitutive formula for fibre section elements to analyze the effect of strain rate effect on the dynamic response of high-speed railway bridges under earthquakes.

2. Concrete strain rate effect

2.1. European Concrete Commission CEB Code

The CEB code gives a dynamic constitutive model of concrete. The applicable range of concrete strain rate is. The compressive, tensile strength, elastic modulus, and the relationship between peak strain and strain rate of concrete are as follows: [13]

1) Compressive strength \( f_{cd} \) of concrete:

\[
\begin{align*}
\frac{f_{cd}}{f_{cm}} &= \left( \frac{\dot{\varepsilon}_c}{\dot{\varepsilon}_co} \right)^{1.026}, & |\dot{\varepsilon}_c| \leq 30s^{-1} \\
\frac{f_{cd}}{f_{cm}} &= \gamma_s \left( \frac{\dot{\varepsilon}_c}{\dot{\varepsilon}_co} \right)^{1/3}, & |\dot{\varepsilon}_c| > 30s^{-1} \\
\end{align*}
\]

(\( \alpha_s = \frac{1}{5+9f_{cm}/f_{cm0}}, \ \lg \gamma_s = 6.156\alpha_s - 2, \ f_{cm} = f_{cm} + \Delta f \))

Where, \( f_{cd} \) is the dynamic strength; \( f_{cm} \) is the static strength; \( \dot{\varepsilon}_c \) is the strain rate; \( \dot{\varepsilon}_co = 3 \times 10^5 s^{-1} \) is the quasi-static strain rate when the concrete is compressed; \( f_{cm0} = 10 \text{ MPa}; \ \Delta f = 8 \text{ MPa} \).

2) Concrete tensile strength \( f_{ctd} \):

\[
\begin{align*}
\frac{f_{ctd}}{f_{ctm}} &= \left( \frac{\dot{\varepsilon}_c}{\dot{\varepsilon}_to} \right)^{1.016\delta}, & |\dot{\varepsilon}_c| \leq 30s^{-1} \\
\frac{f_{ctd}}{f_{ctm}} &= \beta_\delta \left( \frac{\dot{\varepsilon}_c}{\dot{\varepsilon}_to} \right)^{1/3}, & |\dot{\varepsilon}_c| > 30s^{-1} \\
\end{align*}
\]

(\( \alpha_\delta = \frac{1}{10 + 6f_{ctm}/f_{ctm0}}, \ \lg \beta_\delta = 7.112\delta - 2.33, \ f_{cm} = f_{cm} + \Delta f \))

Where, \( f_{ctd} \) is the dynamic strength; \( f_{ctm} \) is the static strength; \( \dot{\varepsilon}_c \) is the strain rate; \( \dot{\varepsilon}_to = 3 \times 10^6 s^{-1} \) is the quasi-static strain rate when the concrete is under tension.

3) Concrete elastic modulus \( E_{cd} \) and peak strain:
\[ \frac{E_{\text{cd}}}{E_{\text{cs}}} = (\dot{\varepsilon}_c / \dot{\varepsilon}_c^0)^{0.026} \]  

(3)

Strain corresponding to peak stress:
\[ \varepsilon_{\text{cd}} / \varepsilon_{\text{cs}} = (\dot{\varepsilon}_c / \dot{\varepsilon}_c^0)^{0.02} \]  

(4)

Where, \( E_{\text{cd}} \) and \( E_{\text{cs}} \) is the strain corresponding to the dynamic and static elastic modulus of the concrete, respectively; \( \dot{\varepsilon}_c \) and \( \dot{\varepsilon}_c^0 \) are the quasi-static strain rate when the concrete is compressed; Strain rate.

Figure 1 shows the dynamic increment coefficients of the concrete parameters recommended by the CEB code, and the abscissa takes the usual logarithmic coordinates.

2.2. Strain rate effect suggested by Mander

Based on experiments, Mander et al. summarized the characteristics of concrete materials under different strain rates [14]. The relationship between concrete strength, elastic modulus, and peak strain and strain rate is as follows:

\[ D_f = \frac{f_{\text{cd}}}{f_{\text{cs}}} = 1 + \frac{\dot{\varepsilon}_c}{0.035 f_c^0} \left[ \frac{f_{\text{cd}}}{0.035 f_c^0} \right]^{0.06} \]  

(5)

\[ D_E = \frac{E_{\text{cd}}}{E_{\text{cs}}} = 1 + \frac{\dot{\varepsilon}_c}{0.035 f_c^0} \left[ \frac{E_{\text{cd}}}{0.035 f_c^0} \right]^{0.06} \]  

(6)

\[ D_{\varepsilon} = \frac{\varepsilon_{\text{cd}}}{\varepsilon_{\text{cs}}} = \frac{1}{3D_f} \left( 1 + \frac{3D_f}{D_E} \right) \]  

(7)

where, \( f_{\text{cd}} \) is the dynamic strength; \( f_{\text{cs}} \) is the static strength; \( \dot{\varepsilon}_c \) is the strain rate; \( E_{\text{cd}} \) and \( E_{\text{cs}} \) are the dynamic and static elastic modulus of the concrete;

Figure 2 shows the changes in strength, elastic modulus, and peak strain with the usual logarithm of the strain rate at that time. It can be seen that the strength and elastic modulus increase slowly with the strain rate, while the peak strain tends to decrease.
2.3. Shiyun Xiao’s test
Shiyun Xiao carried out dynamic tensile tests on a number of concrete test blocks. Statistical analysis of the test data yielded uniaxial tensile strength at different strain rates. The uniaxial dynamic compression test was carried out based on a plurality of 100 × 100 × 300mm test blocks, and the compressive strength and the DIF [3] was obtained, as shown in Fig. 3. The abscissa in the figure is a common logarithmic value of strain rate.

\[
\frac{f_{cd}}{f_{cs}} = 1 + 0.0401 \log \frac{\dot{\epsilon}}{\dot{\epsilon}_0} \tag{8}
\]
\[
\frac{f_{td}}{f_{ts}} = 1 + 0.0571 \log \frac{\dot{\epsilon}}{\dot{\epsilon}_0} \tag{9}
\]
\[
\frac{\epsilon_{cd}}{\epsilon_{cs}} = 0.8133 - 0.03734 \log \dot{\epsilon} \tag{10}
\]

Where: \( f_{cd} \) is the dynamic strength; \( f_{cs} \) is the static strength; \( f_{td} \) is the dynamic tensile strength; \( f_{ts} \) is the static tensile strength; \( \dot{\epsilon} \) is the strain rate; the quasi-static strain rate is taken as \( \dot{\epsilon}_0 = 1 \times 10^{-5} \text{s}^{-1} \); \( \epsilon_{cd} \) and \( \epsilon_{cs} \) are the dynamic and static peak strains of the concrete, respectively.

2.4. Comparison of effects of models
In order to compare the concrete compression curve under the strain rate effect suggested by the above three models, the uniaxial constitutive curves were analyzed with the C35 concrete grade, the axial compressive strength \( f_c = 26 \text{MPa} \), and the stirrup ratio of 0.3%. The uniaxial constitutive of the quasi-static material uses the Mander model, as shown in Figure 4. It can be seen that the difference between the stress-strain curve of each model and the quasi-static uniaxial compression curve becomes more obvious with the increase of the strain rate. The dynamic compression curve suggested by Shiyun Xiao
is smaller than the other two models, and the CEB recommended. The dynamic constitutive curve is not much different from the dynamic constitutive curve suggested by Mander.

\[ \dot{\varepsilon} = 1 \times 10^{-3} \text{s}^{-1} \]

(a) Strain rate \( \dot{\varepsilon} = 1 \times 10^{-3} \text{s}^{-1} \); (b) Strain rate \( \dot{\varepsilon} = 1 \times 10^{-3} \text{s}^{-1} \)

Fig. 4 Stress strain curves comparison of concrete under the three strain rates

3. Rebar strain rate effect

3.1. European Concrete Commission CEB Code

According to the CEB specification, the dynamic strength of the steel bar is obtained from the tensile test data of the steel bar with a yield strength of 420 MPa at a strain rate of \( \leq 10 \text{ s}^{-1} \). The static static strain rate is \( \dot{\varepsilon}_0 = 5 \times 10^{-5} \text{s}^{-1} \), the change law of the yield strength and tensile strength with the strain rate is [13] :

\[
\frac{f_{yd}}{f_{ys}} = 1 + \frac{6}{f_{ys}} \ln \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \tag{11}
\]

\[
\frac{f_{ud}}{f_{us}} = 1 + \frac{6}{f_{us}} \ln \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \tag{12}
\]

In these formulas: \( \dot{\varepsilon} \) is the strain rate; \( \dot{\varepsilon}_0 \) is the strain rate at quasi-static, take \( \dot{\varepsilon}_0 = 5 \times 10^{-5} \text{s}^{-1} \); \( f_{ys}, f_{yd} \) are static and dynamic yield strength respectively; and \( f_{us}, f_{ud} \) are static and dynamic tensile strength respectively.

3.2. Min Li’s test

Min Li conducted uniaxial tensile tests on three types of steel bars, HPB235, HRB335, and HRB400, to study the effects of different strain rates, and obtained the relationship between yield strength and ultimate strength under dynamic strain rates [15], as follows:

\[
\frac{f_{yd}}{f_{ys}} = 1 + c_1 \log \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \tag{13}
\]

\[
\frac{f_{ud}}{f_{us}} = 1 + c_2 \log \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \tag{14}
\]

\[
\frac{\dot{\varepsilon}_{yd}}{\dot{\varepsilon}_{ys}} = 1 + c_3 \log \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \tag{15}
\]

where:

\[
c_1 = 0.1709 - 3.289 \times 10^{-4} f_{ys},
\]

\[
c_2 = 0.02738 - 2.982 \times 10^{-5} f_{ys},
\]

\[
c_3 = 0.9324 - 0.00212 f_{us}
\]

Where, \( \dot{\varepsilon} \) is Strain rate; \( \dot{\varepsilon}_0 \) is quasi-static strain rate, take \( \dot{\varepsilon}_0 = 2.5 \times 10^{-4} \text{s}^{-1} \); \( f_{ys}, f_{yd} \) are static and dynamic yield strength respectively; \( f_{us}, f_{ud} \) are static and dynamic tensile strength respectively; \( \dot{\varepsilon}_{us} \).
and $\varepsilon_{id}$, respectively, is initial strain for static and dynamic strain hardening; $c_f$, $c_u$, and $c_h$ is a parameter expressed by the static yield strength.

The CEB code does not differ much from the dynamic increase coefficient at the strain rate suggested by Li Min. Among them, the DIF of yield strength is generally larger than the DIF of tensile strength, which is between 1.0 and 1.2. The increase factor is between 1.0 and 1.1.

4. Effect of strain rate on bridge response under earthquake

4.1. Numerical simulation of fibre section beam element method

After the structure is divided into units along the axial direction, the section is divided into fibres with smaller areas, and the uniaxial material constitutive curve is given to each fibre. Through integral calculation of the cross section, the bending moment-curvature curve of the cross section can be obtained, and the stiffness matrix and flexibility matrix of the cross section can be determined, and then the element stiffness matrix in the local coordinate system can be obtained, and finally converted to the overall coordinate system for solution [16].

The bridge pier model is simulated using fibre beam elements. In order to accurately simulate the load-displacement curve of the bridge pier, a modified Chang-Mander concrete uniaxial constitutive model is used, and the uniaxial constitutive model of the reinforced bar adopts the Giuffré-Megotto-Pinto model. The modified Chang-Mander model and the reinforced Giuffré-Menegotto-Pinto model can better simulate the hysteretic response of the bridge pier at low longitudinal reinforcement ratio. The author compared and studied a variety of concrete uniaxial constitutive models in Reference 2. The results show that the simulation results of the two constitutive models are in good agreement with the experimental data.

1) Modified Chang-Mander model:

After Chang and Mander carried out a large number of pier column tests, they improved on the basis of the original Mander model to obtain the uniaxial constitutive curve of concrete [17]. Based on experiments, Waugh further modified the Chang-Mander model [18], and simplified the unloading curve to a three-segment straight line for the sake of calculation, ensuring the stability of the calculation and easy convergence. The compression skeleton curve is divided into two parts, unconstrained and constrained concrete. The uniaxial curve of unconstrained concrete is expressed as follows:

$$x = \frac{\varepsilon_c}{\varepsilon_{o}} \quad y = \frac{\sigma_c}{f'_{c}} \quad y = \frac{nx}{1 + (n - \frac{r}{r - 1})x + \frac{x^r}{r - 1}}$$

$$r = f'_{c}/5.2 - 1.9, \quad n = E_c\varepsilon_{o}/f'_{c}, \quad E_c = 8200(f'_{c})^{0.8}, \quad \varepsilon_{o} = (f'_{c})^{0.8}/1153$$

where, $\sigma_c$ and $\varepsilon_c$ are the stress and strain of the concrete, respectively; $f'_{c}$ and $\varepsilon_{o}$ are the peak stress and peak strain of the concrete, respectively; $E_c$ is the initial elastic modulus of the curve.

2) Reinforced uniaxial constitutive model

The Giuffré-Menegotto-Pinto model can better describe the hysteretic behavior of steel bars under cyclic loading, and has been verified in multiple experiments. The following equation is used to describe the model's stress-strain curve [19]:

$$\frac{\sigma - \sigma_t}{\sigma_t - \sigma_i} = \left[ \frac{E_{bh}}{E_i} + \frac{1 - E_{bh}/E_i}{1 + \varepsilon^R} \right] \varepsilon \quad (R = R_o - a_2 \frac{\varepsilon}{a_1 + \frac{\varepsilon}{\varepsilon_{max} - \varepsilon_{i}}} \frac{\varepsilon_{max} - \varepsilon_{i}}{\varepsilon_{i} - \varepsilon_{i}}, \quad \varepsilon = \frac{\varepsilon_{max} - \varepsilon_{i}}{\varepsilon_{i} - \varepsilon_{i}})$$

where, $\varepsilon_{max}$ is the maximum process strain experienced by $\varepsilon_{i}$; the variable R specifies the Bauschinger effect; the parameters $R_o$, $a_1$, and $a_2$ are determined through experimental tests.
4.2. Model analysis

The effect of strain rate on the structural response is firstly manifested by changes in the properties of materials such as concrete and steel bars, thereby causing changes in structural response such as structural displacement and internal forces. The length of high-speed railway bridges is generally several kilometres or even tens of kilometres. Three-span 32m simply-supported box girder is selected, and 4 bridge piers are selected as the calculation model. The pier height is 16m, and the second-phase dead load of the bridge is 120kN/m.

Side piers and semi-span beams are treated with concentrated mass on the piers, and each bridge pier is connected with the bridge piers and box girder by setting basin-type rubber bearings. A spring element in the longitudinal bridge direction is set between the box girder to simulate the bridge system stiffness of the track plate and steel rail on the beam. The value of the spring stiffness of the bridge system \( K_q = 1.0353 \times 10^7 \text{kN/m} \) [20]. In the calculation model, the bridge piers and box girder use fibre beam and column elements, while the bearings use non-linear double-fold spring elements. The damping adopts Rayleigh damping. The calculation prototype and finite element model of the simply-supported beam bridge are shown in Figures 5 and 6.

![Fig. 5 Prototype of simply supported beam bridge for high-speed railway](image)

![Fig. 6 Finite element model of simply supported girder bridge for high-speed railway](image)

The concrete strength grade of the simply-supported box girder is C50, the concrete strength grade of the pier is C35, the quasi-static axial compressive strength of the concrete is \( f_c = 26 \text{ MPa} \), the longitudinal reinforcement is HRB335, the static yield strength and static tensile strength of the reinforcement is 380 MPa and 540 MPa respectively, and the longitudinal reinforcement ratio is 0.4%. The cross section of piers is shown in figure 7.

![Fig. 7 Schematic diagram of pier section (unit: cm)](image)

The seismic fortification intensity of the bridge is set to 8 degrees, the horizontal acceleration of the design earthquake is 0.2g, and the horizontal peak ground acceleration (PGA) of rare earthquakes is 0.38g. The El-Centro seismic wave was used, and the seismic effects in the longitudinal, transverse,
and vertical directions were also considered. The peak acceleration ratio in the three directions was 1: 0.85: 0.65, and the peak ratio of the original seismic wave was adjusted.

In the analysis, the concrete uniaxial material constitutive model adopts the aforementioned modified Chang-Mander model, and the reinforcing bar adopts the Giuffré-Megotto-Pinto model, and the aforementioned fibre beam-column element method is used for numerical analysis using OpenSees. The DIF of the strain rate of the concrete material is recommended by the CEB code, and the DIF of the strain rate of the properties of the reinforced material is the formula suggested by Li Min. The material characteristics at different strain rates are listed in Table 1.

Figures 8 and 9 show the strain rate time-history curves of the steel bars located at the bottom of the pier and the top of the pier. The strain rate of the material gradually decreases along the height of the pier. The strain rates of the steel bars in both directions of the pier bottom section are within $4 \times 10^{-2}$ s$^{-1}$, while the strain rates of the steel bars in both directions of the pier top section are both within $8 \times 10^{-4}$ s$^{-1}$, the amplitudes of the strain rates in the two directions are not significantly different. Due to the large internal force at the bottom of the pier, the stress and strain at the bottom of the pier are large, and the strain changes are also the most severe.

### Table 1 Material characteristic parameters under different strain rates

| Strain rate $/ (s^{-1})$ | Concrete material properties | Rebar material properties |
|-------------------------|------------------------------|----------------------------|
|                         | $f_c$/MPa | $f_t$/MPa | $E_c$/MPa | $f_y$/MPa | $f_u$/MPa |
| $1.0 \times 10^{-5}$    | 26.0      | 2.34      | 28209     | 380       | 540       |
| $1.0 \times 10^{-4}$    | 26.9      | 2.43      | 28209     | 384       | 542       |
| $1.0 \times 10^{-3}$    | 28.4      | 2.56      | 28759     | 398       | 545       |
| $1.0 \times 10^{-2}$    | 30.5      | 2.69      | 29535     | 411       | 551       |
| $1.0 \times 10^{-1}$    | 32.8      | 2.93      | 30615     | 425       | 560       |

Note: $f_c$ is the axial compressive strength; $f_t$ is the axial tensile strength; $E_c$ is the initial elastic modulus; $f_y$ is the yield strength of the steel bar; $f_u$ is the tensile strength of the steel bar.

![Fig. 8](image1.png)  
(a) longitudinal; (b) transverse

Fig. 8 The strain rate time history of the outside steel rebar at the bottom of pier

![Fig. 9](image2.png)  
(a) longitudinal; (b) transverse

Fig. 9 The strain rate time history of the outside steel bar at the top of the pier
Figure 10 shows the moment-curvature curves of the pier bottom in two directions, showing that the pier bottom start yielding, and the plasticity development of the transverse bridge is slower than the forward direction. This shows that under the action of a rare earthquake (PGA 0.38g), the pier enters the plastic stage, the pier is damaged, there is still a large energy consumption reserve, and the pier will not collapse.

![Moment-curvature curves](image)

Fig. 10 Moment-curvature curves at the bottom of pier analyzed by the strain rate $1.0 \times 10^{-5}$

According to the comparison of the bending moments and curvature curves of the pier bottom calculated using the material characteristics under different strain rates, as shown in Figures 11-12. When the seismic load is the same, the maximum time history curvature of the pier bottom and the pier top calculated according to the quasi-static material characteristics of the bridge pier. The maximum time history displacement is larger, and it is easier to enter yield, but it has less effect on the result.

In order to compare the change of the strain rate of the pier under different PGA, the pier response when the horizontal PGA is 0.64g according to a rare earthquake of 9 degrees is calculated. The PGA ratio of the three directions, horizontal, and vertical directions is still set to 1:0.85:0.65. Figure 13 shows the strain rate time history of the outermost concrete at the bottom of the pier and the top of the pier. The strain rate of the pier bottom section reaches $0.1 \text{s}^{-1}$, and the pier top section strain rate reaches $1.3 \times 10^{-2} \text{s}^{-1}$. The strain rate has increased significantly compared to the 8-degree fortification earthquake.

![Strain rate](image)

Fig. 11 Moment-curvature curves at the bottom of pier analyzed according to two kinds of strain rate materials
Fig. 12 The displacement time histories at the top of pier analyzed by two kinds of strain rate materials

In order to compare the effect of different PGA on the strain rate effect on the dynamic response of the bridge pier, the moment-curvature curve at the bottom of the pier when the PGA are 0.64g and 0.80g is analyzed, as shown in Figure 14. For earthquakes in three directions, longitudinal, transverse and vertical, the three-way acceleration peak ratio is still set to 1: 0.85: 0.65. It can be seen that as the peak acceleration increases, the difference between the curve calculated at the quasi-static material strain rate of $1.0 \times 10^{-5} \text{s}^{-1}$ and the material constitutive curve at the strain rate of 0.1/s is more and more obvious, indicating that the bridge When the peak earthquake acceleration is large, the strain rate effect needs to be accurately considered and cannot be ignored.

(a) PGA of 0.64g;  (b) PGA of 0.80g

Fig. 13 The time history of strain rate at the top and bottom of the pier under the conditions of earthquake PGA of 0.64g

(a) section at the bottom of pier;  (b) section at the top of pier

Fig. 14 The moment curvature curves at the bottom of pier analyzed by two different strain rate materials under the earthquake loading with PGA
4.3. Relevant research
Zhang Hao analyzed the dynamic time-history response of a multilayer concrete shear wall structure under earthquake and found that the displacement response when the strain rate effect is not considered is greater than the displacement when the strain rate is considered, and as the peak acceleration of the earthquake increases, the effect of rate on structural response will also increase [21]. Long Yeping carried out dynamic cyclic load tests on reinforced concrete columns and found that the horizontal force of the columns decreased rapidly and the ductility decreased after reaching the failure load after increasing the strain rate [22]. Both of them are similar to the research results of this article in both theoretical analysis and experiment.

5. conclusion
In this paper, the strain rate of the pier material under earthquake and the effect of the strain rate effect on the dynamic response of high-speed railway pier are studied, and the following conclusions are obtained:
1) The strength of concrete and steel bars generally increases at high strain rates. The strain rates of the materials in the two directions on the pier bottom section are generally within $1 \times 10^{-1}/s$, and the amplitudes of the strain rates in the two directions are not much different.
2) Under the same seismic load, the maximum moment of the pier bottom calculated from the quasi-static material properties of the bridge pier is smaller than the bending moment calculated from the material properties under the consideration of the strain rate, which will be unsafe during design. The maximum displacement of the pier is also larger.
3) The strain rate effect of the pier before the damage load has a small effect on the results. Dynamic analysis can be performed using the static test results and theoretical analysis methods of this type of pier. When collapse is possible, the dynamic response of the bridge caused by the strain rate effect is obvious and cannot be ignored.

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Reference
[1] National development and reform commission of China. Medium- and long-term railway network planning[R]. Beijing: National development and reform commission, 2016:1–4.
[2] SHAO G, JIANG L CHOUW N. Experimental investigations of the seismic performance of bridge piers with rounded rectangular cross-sections[J]. Earthquakes and Structures, 2014, 7(4): 463–484.
[3] XIAO Shiyun. Rate-dependent Constitutive Model of Concrete and Its Application to Dynamic Response of Arch Dams[D]. Dalian: Dalian University of Technology,2002.
[4] ROSS C A, TEDESCO J W, KUENNEN S T. Effects of strain rate on concrete strength[J]. ACI Materials Journal, 1995, 92 (1): 37–47.
[5] PAL N. Seismic cracking of concrete gravity dams[J]. Journal of the Structural Division-ASCE,1976, 102(9):1827–1844.
[6] AHMAD S H, SHAH S P. Behavior of hoop confined concrete under high strain rates[J]. Journal of the American Concrete Institute, 1985, 82(5): 634–647.
[7] ROSS C A, THOMPSON P Y, TEDESCO J W. Split-hopkinson pressure-bar tests on concrete and mortar in tension and compression[J]. ACI Materials Journal, 1989, 86(5): 475–481.
[8] KULKARNI S M, SHAH S P. Response of reinforced concrete beams at high strain rates[J]. ACI Structural Journal, 1998, 88(3): 705–715.
[9] XU Bin, LONG Yeping. Study on the behavior of reinforced concrete columns with fibre model considering strain rate effect[J]. Engineering Mechanics, 2011, 28(7): 103–108.
[10] BISCHOFF P H, PERRY S H. Compressive behaviour of concrete at high strain rates [J]. Materiaux et constructions, 1991, 24(144): 425-450. KULKARNI S M, SHAH S P. Response of reinforced concrete beams at high strain rates [J]. ACI Structural Journal, 1998, 88(3): 705-715.

[11] ROSSI P, TOUTLEMOBDE F. Effect of loading rate on the tensile behavior of concrete: Description of the physical mechanisms [J]. Materials and Structures/Materiaux et Constructions, 1996, 29(186): 116-118.

[12] MALVAR L J, ROSS C A. Review of strain rate effects for concrete in tension [J]. ACI Materials Journal, 1998, 95(6): 735-739.

[13] CEB-FIP model code 1990, Model code for concrete structures [S]. London: Thomas Telford, 1993.

[15] KULKARNI S M, SHAH S P. Response of reinforced concrete beams at high strain rates [J]. ACI Structural Journal, 1998, 88(3): 705-715.

[14] MANDER J B, PRIESTLEY M J N, Park R. Theoretical stress-strain model for confined concrete [J]. Journal of Structural Engineering, ASCE, 1988, 114(8): 1804-1825.

[15] LI Min, LI Hongnan. Dynamic test and constitutive model for reinforcing steel [J]. China Civil Engineering Journal, 2010, 43(4): 70-75.

[16] TAUCER F F, SPACONE E, FILIPPOU F C. A fibre beam-column element for seismic response analysis of reinforced concrete structures [R]. Berkeley: Earthquake Engineering Research Center, University of California, Berkeley, 1991: 10-20.

[17] CHANG G A, MANDER J B. Seismic energy based fatigue damage analysis of bridge columns: part I–evaluation of seismic capacity [R]. Buffalo: State University of New York, Buffalo, 1994: 9-20.

[18] WAUGH J. Nonlinear analysis of T-shaped concrete walls subjected to multi-directional displacements [D]. Ames: Iowa State University, 2009.

[19] FILIPPOU F C, POPOV E P, BERTERO V V. Effects of bond deterioration on hysteretic behavior of reinforced concrete joints [R]. Berkeley: Earthquake Engineering Research Center, University of California, Berkeley, 1983: 29-35.

[20] CHEN Xuexi, ZHU Xi, GAO Xuekui. Analysis on the pounding responses between adjacent bridge beams under earthquakes [J]. China Railway Science, 2005, 26(6): 77-81.

[21] ZHANG Hao. Strain rate effect of materials on seismic response of reinforced concrete frame-wall structure [D]. Dalian: Dalian University of Technology, 2012.

[22] LONG Yeping. Experimental Study on Dynamic Hysteretic Rule of Reinforced Concrete Columns under Rapid Loading [D]. Changsha: Hunan university, 2010.