Effects of near-fault pulse-type ground motions on high-speed railway simply supported bridge and pulse parameter analysis

Tuo Zhou1 · Lizhong Jiang1,2 · Ping Xiang1 · Zhipeng Lai1 · Yuntai Zhang1 · Xiang Liu3

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Abstract
Based on an innovative high-speed railway simply supported bridge (HSRSSB) model, the potential influence of near-fault pulse-type ground motions on HSRSSB is analyzed. Eighty-five original records of such ground motions are imposed on the simulation model. The dynamic responses of the bridge are calculated, including girder displacement, rail displacement, pier bending moment, bearing deformation, and residual displacement. Linear regression analysis reveals the relationship between the dynamic response and intensity measures (IMs). Moreover, six typical ground motions are chosen from these eighty-five records as prototypes in order to perform a pulse parameter analysis. Through an artificial ground motion synthesis, the effects of parameters, such as pulse amplitude, pulse period, and pulse number, on the behavior of HSRSSB are investigated and discussed. It is concluded that the response of HSRSSB depends on the acceleration-type IMs and presents a strong linear relationship with spectral IMs at the fundamental period of the structure. Regarding the pulse parameters, the HSRSSB dynamic response shows a positive correlation relationship with pulse amplitude, but a negative correlation with pulse period.

Keywords High-speed railway · Simply supported bridge · Seismic excitation · Near-fault ground motion · Velocity pulse effect

1 Introduction

High-speed railway (HSR) is prevailingly utilized all over the world due to the advantages of convenience and high-efficiency. Nevertheless, potential risks caused by external excitation may arise during its construction and operation procedure. With the wide distribution of the HSR network, many bridges located near seismic zones are threatened by near-fault
earthquake events. High-speed railway simply supported bridges (HSRSSB) constitute a significant alternative to ground alignment in HSR design as they serve the concept of replacing roads with bridges. However, their safety against near-fault earthquakes becomes a great challenge from design and operation perspectives.

In general, the near-fault earthquakes are seismic events with a ruptured fault distance not exceeding 10 km (Mavroeidis 2003) to 20 km (Bray and Rodriguez-Marek 2004). Furthermore, the near-fault earthquakes have special characteristics, including the effects of hanging wall and footwall (Abrahamson and Somerville 1996), ruptured fault directivity (Bray and Rodriguez-Marek 2004; Howard et al. 2005), and velocity pulse (Farid Ghahari et al. 2010). Through the investigation of Parkfield ground motion, recorded on June 27, 1966, which caused severe damages to adjacent structures as it was a near-fault pulse-type ground motion, Housner and Trifunac (1967) concluded that the design of important structures, under the risk of ground motions similar to Parkfield earthquake event, requires special attention.

Over the years, scholars have revealed the mechanism of the velocity pulse of near-fault ground motions (Kawase and Aki 1990; Bouchon et al. 2001). Generally speaking, the near-fault pulse-type ground motion can be classified into 2 kinds according to the velocity pulse, either with forward directivity or with permanent translation (Somerville et al. 1997; Mavroeidis 2003). Based on the classification of the genetic principle of pulse-type earthquakes, the analysis and simulation of them are carried out accordingly. Alavi-Shushtari (2001) utilized the parameters of pulse period and intensity to simulate different pulses. Combined with pulse shapes that consisted of half-pulse, full pulse, and multiple pulses, the elastic strength spectrum of different artificial ground motions could be obtained. In the pulse model, proposed by Menun and Fu (2002), the shape parameters were introduced, meaning the vector of parameters with pulse amplitude, period, and the beginning time. Through this model, the authors preliminarily predicted the structural response arisen by near-fault pulse-type ground motions. Replacing the Gaussian envelope of the Gabor wavelet (Gabor 1946) with a shifted haversed sine function, Mavroeidis (2003) set up a simulation model to define the period of the velocity pulse, obtaining the model calibration by simulating typical pulse-type ground motion records, including “Parkfield”, “San Fernando”, “ChiChi”, etc. The aforementioned research provided a premise for investigation of the structural response under near-fault pulse-type ground motions. The dynamic response of a single degree of freedom system (Alavi-Shushtari 2001; Chopra and Chintanapakdee 2001) revealed the basic principle that velocity pulses lead to larger deformations and internal forces of the structure. For a multi-story structure, the story displacement ductility is significantly influenced by the period and amplitude of the pulse, and when the ratio between the periods of pulse and structure ranges within 0.5–2.5, the dynamic response of the structure is dominantly controlled by the velocity pulse (Li et al. 2017). Phan et al. (2007) applied “Rinaldi” and “El Centro” ground motions to reinforced concrete bridge columns, and concluded that the velocity pulse leads to large irreversible displacements of the bridge columns in the direction of the velocity pulse. Li et al. (2017) utilized artificial pulse-type ground motions, based on the pulse simulation method proposed by Mavroeidis (2003), and studied the effect of different pulse parameters on super-span cable-stayed bridge systems. In that paper of Li et al. (2017), the cutoff frequency of near-fault records was calculated by an equation that contained empirical coefficients, which led to potential insufficient filtering. In order to obtain target frequency with accuracy, Xu and Chen (2021) proposed simplified methods of digital ground motions filtering both high and low frequencies. Zhang et al. (2020) discussed the dynamic response of simply-supported highway bridges. Their research results indicated that with the increase of pulse amplitude
and period, the bridge response, including pier deformation and pier-girder relative displacement, increased.

Nowadays, academic research of the safety of various bridges under different kinds of excitations (Nielson and DesRoches 2007; Zhu et al. 2020; Guo et al. 2020b; Lai et al. 2020; Homaei and Yazdani 2020; Jahangiri and Yazdani 2021; Lai et al. 2021), and the train running safety of HSRSSB under earthquake (Du et al. 2012; Guo et al. 2021; Liu, et al. 2021a, b; Montenegro et al. 2016, 2021) have become mature. However, the research that takes into account near-fault ground motions with or without velocity pulse is still insufficient. Guo et al. (2020a) found that the sliding layer and shear alveolar are the most vulnerable components of HSRSSB, and concluded that the dynamic response is governed by high-frequency part of near-fault seismic excitation. After studying the failure mode of bearings and pier shear capacity subjected to near-fault ground motions, Jiang et al. (2020) suggested that the strength of HSRSSB structure requires special design due to the high possibility of failure under such ground motions. Chen et al. (2013) found that the pier top displacement, and bending moment of pier bottom increased due to the existence of velocity pulse, and the damage of the structure was controlled by pulse-velocity.

On the premise of these studies, in order to further explore the influence of near-fault pulse-type ground motions on the HSRSSB, eighty-five near-fault ground motions with pulse effects are applied as excitation on an innovative HSRSSB model. The relationship between different intensity measures (IMs) and HSRSSB dynamic response is discussed. Moreover, six representative ground motions are selected for the impact of pulse parameters on HSRSSB.

2 HSRSSB model introduction

The prototype of the HSRSSB model is shown in Fig. 1. A typical HSRSSB consists of longitudinal components, interlayer components, and piers. The main components are piers, girders, track system, and rails. The China Railway Track System (CRTS II) is taken as an example, which includes track plate, and base plate, while the interlayer components are fastener, CA mortar layer, sliding layer, etc., as listed in Table 2. The geometry parameters and cross-section information of girder and pier of the prototype can be found in Table 1 and Fig. 2. The length of the girder and the height of the piers are 32.25 m and 10 m, respectively. The geometry parameters of other longitudinal components are listed in Table 1, as well.
The model of HSRSSB is based on the finite-element (FE) theory. The track system is modeled as discrete 4-layer FE beam element, as shown in Fig. 3. The element length of the longitudinal components, including rail, track plate, base plate, and girder, is 0.645 m. The interlayer components listed in Table 2 are considered as springs which provide

### Table 1  Geometry parameters of the bridge components (Zhou et al. 2022)

| Components          | Cross-sectional Area (m²) | Iₓ (m⁴) | Iᵧ (m⁴) | Iₓₓ (m⁴) | Thickness/Depth (m) | Width (m) |
|---------------------|---------------------------|---------|---------|---------|---------------------|-----------|
| Rail                | 0.008                     | 5.20 × 10⁻⁶ | 3.19 × 10⁻⁵ | 2.32 × 10⁻⁶ | 0.1738              | 0.10      |
| CRTS II track plate | 0.510                     | 0.276   | 0.0017  | 0.0065  | 0.20                | 2.55      |
| Base plate          | 0.561                     | 0.406   | 0.0017  | 0.0065  | 0.19                | 2.95      |
| Girder              | 12.831                    | 107.157 | 15.128  | 31.693  | 3.05                | 13.40     |
| Pier                | 21.962                    | 101.905 | 15.226  | 49.235  | 3.00                | 8.00      |

![Fig. 2 Cross-section of girder and pier](image)

![Fig. 3 Modeling of 4-layer HSRSSB (Zhou et al. 2022)](image)
stiffness to the track system. The interval of springs, representing fasteners, CA mortar layer, and sliding layer, is the same as the element length. The location of the shear alveolar is assumed to be at the end of each girder, while the interval of shear bar and lateral choke block is set as 5.16 m. The response in longitudinal direction of HSRSSB is mainly investigated and discussed due to vulnerability of HSRSSB components to the seismic excitation in longitudinal direction (Shao et al. 2014; Guo et al. 2020b). An elastic–plastic constitutive model of interlayer components is set for longitudinal direction, while the springs in Y-direction and Z-direction are set as linear. The force–displacement relationship of elastic–plastic springs is given in Fig. 4, the force and displacement values at yield point are listed in Table 2, where the linear stiffness of other two direction are also given.

In order to enhance the computational efficiency, an innovative simplified model of HSRSSB called consistent boundary model (CBM) is introduced and subjected to seismic excitation. Most conventional models on the ANSYS platform simulate an integrated HSRSSB with all spans, approaches and abutments that correspond to realistic boundary conditions. Thus, if the prototype HSRSSB has a large number of spans, the model of the bridge can be particularly complicated due to many degrees of freedom, while the computational efficiency decreases as a result of large stored model data. The CBM illustrated in Fig. 5 consists of only four spans of four-layer beams. The boundary conditions of CBM are different with respect to the conventional model: the restraints at both sides of the bridge provided by abutments and approaches are replaced by couplings between the start and end of the track system. Figure 5 also demonstrates the details of the coupling procedure. Extra pseudo elements of rail, CRTS II track plate, and

### Table 2 Stiffness of connector components (Zhou et al. 2022)

| Components            | F' (kN) | d' (mm) | k' (kN/m) | k'' (kN/m) | Y-axis(kN/m) | Z-axis(kN/m) |
|-----------------------|---------|---------|-----------|------------|--------------|--------------|
| Fastener              | 8.65    | 2.00    | 4.31 × 10³| 0          | 4.33 × 10³   | 1.38 × 10⁴   |
| CA mortar layer       | 45.05   | 0.50    | 9.00 × 10⁴| 0          | 9.00 × 10⁴   | 2.00 × 10⁸   |
| Sliding layer         | 6.00    | 0.50    | 1.20 × 10⁴| 0          | 1.20 × 10⁴   | 1.38 × 10⁵   |
| Lateral choke block   | 6.00    | 0.50    | 1.20 × 10⁴| 0          | 2.39 × 10⁵   | 1.38 × 10⁷   |
| Shear bar             | 180.00  | 0.075   | 2.40 × 10⁵| 0          | 2.40 × 10⁶   | 2.00 × 10⁹   |
| Shear alveolar        | 1.20 × 10⁵| 0.12   | 1.00 × 10⁹| 0          | 1.00 × 10⁹   | 1.38 × 10⁹   |
| Movable bearing       | 51.60   | 2.00    | –         | –          | 2.58 × 10⁴   | 1.00 × 10⁷   |
| Fixed bearing         | 5.16 × 10³| 2.00    | 2.58 × 10⁶| 2.58 × 10⁴| 2.58 × 10⁶   | 1.00 × 10⁷   |

Fig. 4 Force–displacement curves of longitudinal interlayer components
and base plate are set at the right side of the 4th span, coupled with the corresponding ones at the left side of 1st span. The pier at the right side of the 4th span is omitted, and the bearing is directly coupled with the left pier of the 1st span. In this way, the boundary conditions of all four spans in CBM are consistent. In order to validate the feasibility and accuracy of CBM, a comparison between numerical modeling and a shake table test is conducted. The HSRSSB is 49-span simply-supported bridge with parameters given in Table 1 and Table 2. The dynamic response of CBM model is compared with the results of the middle span (the 25th span) of the 49-span HSRSSB. The detailed results can be found in the research of Zhou et al. (2022). The consistency of the 1st mode coordinates and the 1st frequencies of the two models, i.e., 4.1349 Hz and 4.1594 Hz, for CBM and conventional model, respectively, leads to the conclusion that for 10 m pier height, the CBM model can accurately simulate the dynamic response at mid-span of HSRSSB with forty-nine spans.

In order to further verify the accuracy of CBM four earthquake records listed in Table 3 are imposed as longitudinal excitation on the simulated CBM model and the conventional model. Figure 6 demonstrates the comparison results, by means of partial time-history response of the mid-span girder displacement under these four seismic excitations, showing a satisfactory agreement. Moreover, the maximum differences of these responses for the four earthquakes arise at 1.25%, 1.19%, 2.05%, and 3.32%, respectively. These small errors that do not exceed 5% also verify the feasibility and accuracy of CBM.

**Table 3** Earthquake events for CBM validation

| Earthquake name     | Year | Station                        | Magnitude |
|---------------------|------|--------------------------------|-----------|
| Lytle Creek         | 1970 | Colton—So Cal Edison           | 5.33      |
| Imperial Valley-02  | 1940 | El Centro Array #9             | 6.95      |
| Parkfield           | 1966 | Cholame—Shandon Array #8       | 6.19      |
| San Fernando        | 1971 | Fairmont Dam                   | 6.61      |
3 IMs investigation

3.1 Selection of near-fault pulse-type ground motions

Due to the insufficiency of seismic records of typical near-fault motion with pulse effect, in this paper, eighty-five pulse-type ground motions with rupture distance $R_{rup}$ ranging from 0 to 30 km are selected and imposed as longitudinal seismic excitations on the ANSYS CBM model. All these records are taken from PEER Database NGA-West2 (PEER website), which include typical near-fault pulse-type ground motions such as the Chi-Chi earthquake, Imperial Valley earthquake, Loma Prieta earthquake, etc., and cover the magnitude range from 5.74 to 7.9 as shown in Fig. 7.

Fig. 6 Comparison of displacement of girder at mid-span between CBM and conventional model, a Lytle Creek, b Imperial Valley-02, c Parkfield, d San Fernando
3.2 Correlation sensitivity analysis

In seismic engineering, ground motion IMs are significantly important parameters for fragility analysis and reliable probabilistic seismic demand analysis (PSDA) (Jahangiri et al. 2018). The most common IMs such as peak ground acceleration (PGA) and acceleration of response spectrum at fundamental period of structure (Sa-T1) are widely utilized for the assessment of bridge vulnerabilities (Nielsen and DesRoches 2007). Besides, researches also reveal that IMs based on time-history velocity and response spectrum can be more suitable for some specific bridge structures such as extended pile-shaft-supported bridges and cable-stayed bridges (Wang et al. 2018; Zelaschi et al. 2019; Wei et al. 2020). Different with common earthquakes, near-fault pulse-type ground motion can lead to larger structural response and failure controlled by pulse velocity. In order to understand the relationship between IMs of near-fault pulse-type ground motions and dynamic response of HSRSSB, correlation sensitivity analyses of sixteen different IMs are conducted and 4 kinds of bridge response results are calculated. The IMs being investigated in this paper are listed in Table 4, and the bridge response results are the maximum value of time-history response of displacement of girder at mid-span ($\varepsilon_g$), displacement of rail at mid-span ($\varepsilon_r$), bending moment of pier bottom at middle span ($\varepsilon_m$), and fixed bearing deformation at middle span ($\varepsilon_b$).

The linear regression analysis based on Eq. (1), assuming that the relationship between bridge response and IMs, is exponential, where $\alpha$ is the regression coefficient reflects correlation sensitivity between IMs and bridge response.

$$\ln(\varepsilon) = \alpha \cdot \ln(IMs) + \ln(b)$$  \hspace{1cm} (1)

Figure 8 demonstrates the linear regression results of $\ln(IMs)$-$\ln(\varepsilon_g)$. It can be found that the PSA has the largest $\alpha$ value, 1.03797. This means that the maximum value of girder displacement at mid-span is the most sensitive to PSA. For other IMs, $\varepsilon_g$ is also sensitive to $S_{a-T1}$, $S_{v-T1}$, $S_{d-T1}$, and PGA, for the $\alpha$ values of all these five IMs exceed 0.9. Moreover, the correlation between $\varepsilon_g$ and IMs of PGD, PSD, and SED are not sensitive. Only the diagram of $\varepsilon_g$ is demonstrated in Fig. 8, while the linear regression results of the other results mentioned above are listed in Table 5. The parameter for consistency assessment of sample points and linear regression function, denoted as $R^2$, which is called adjusted
coefficient of determination, is expressed by Eq. (2), where TSS is the total sum of square, and RSS is the residual sum of square.

\[ R^2 = 1 - \frac{RSS/df_{\text{Error}}}{TSS/df_{\text{Total}}} \]  

As Table 5 demonstrates, the top five sensitive IMs for rail displacement at mid-span \( \varepsilon_r \) are PSA, \( S_{a-T1} \), \( S_{d-T1} \), \( S_{a-0.2} \), and PGA, which is similar to the case of \( \varepsilon_g \). The most sensitive IMs for the bending moment of the pier bottom \( \varepsilon_m \) and bearing displacement \( \varepsilon_b \) is PSA as well, with \( a \) values reaching 0.68525 and 1.82233, respectively. In general, all these 4 structure responses (\( \varepsilon \)) are sensitive to acceleration-type IMs including PSA, PGA, \( S_{a-0.2} \), and \( S_{a-T1} \). However, IM \( S_{a-1.0} \) has small \( a \) values compared with other acceleration-type IMs. The explanation is that the fundamental period of HSRSSB model is only 0.242 s. For velocity-type IMs, structural responses are only sensitive to \( S_{v-T1} \), while the \( a \) values of PGV and PSV are relatively low. In PSDA, the aforementioned \( a \) value is the main measure of optimal IM criteria, PRACTICALITY. IMs with higher \( a \) value indicates that the structure responses are more sensitive to the IMs as illustrated above, and the IMs are more practical. Combined with Table 5, acceleration-type IMs tend to be more practical, while PGD and PSD perform the worst practicality.

As \( R^2 \) reflects the consistency of sample points and linear regression function, higher \( R^2 \) value indicates better degree of linear regression fit. Based on Table 5, it can be noted that IM \( S_{d-T1} \) presents the best linear regression consistency for all four structural responses (\( \varepsilon \)) due to the highest \( R^2 \) value, whilst, spectral acceleration (i.e., \( S_{a-T1} \)) and velocity at fundamental period (\( S_{v-T1} \)) of structure have relatively strong linear regression correlation with \( \varepsilon \). Hence, for near-fault pulse-type ground motions, spectral acceleration, velocity,
Fig. 8  Linear Regression of girder displacement at mid-span ($x_g$) and IMs
### Table 5 Linear regression results

| $\epsilon$ | $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ | $R^2_g$ | $R^2_r$ | $R^2_m$ | $R^2_b$ |
|------------|--------------|--------------|--------------|--------------|---------|---------|---------|---------|
| $a$        | 0.90883      | 0.90886      | 0.60256      | 1.58149      | 0.78391 | 0.7838  | 0.80231 | 0.72693 |
| $\beta_y$  | 0.28917      | 0.28927      | 0.80231      | 0.58708      | 0.28917 | 0.28927 | 0.80231 | 0.58708 |

IM

| $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ |
|--------------|--------------|--------------|--------------|
| 0.28917      | 0.28927      | 0.80231      | 0.58708      |

| $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ |
|--------------|--------------|--------------|--------------|
| 0.28917      | 0.28927      | 0.80231      | 0.58708      |

| $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ |
|--------------|--------------|--------------|--------------|
| 0.28917      | 0.28927      | 0.80231      | 0.58708      |

| $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ |
|--------------|--------------|--------------|--------------|
| 0.28917      | 0.28927      | 0.80231      | 0.58708      |

| $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ |
|--------------|--------------|--------------|--------------|
| 0.28917      | 0.28927      | 0.80231      | 0.58708      |

| $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ |
|--------------|--------------|--------------|--------------|
| 0.28917      | 0.28927      | 0.80231      | 0.58708      |

| $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ |
|--------------|--------------|--------------|--------------|
| 0.28917      | 0.28927      | 0.80231      | 0.58708      |

| $\epsilon_g$ | $\epsilon_r$ | $\epsilon_m$ | $\epsilon_b$ |
|--------------|--------------|--------------|--------------|
| 0.28917      | 0.28927      | 0.80231      | 0.58708      |
and displacement at fundamental period of structure have strong linear positive correlation with the structural responses mentioned in this section. Moreover, the evaluation criterion reflecting the degree of correlation between sample points and linear regression function in PSDA is EFFICIENCY, which is controlled by logarithmic standard deviation $\beta_{e|IM}$ given by Eq. (3). The values of $\beta_{e|IM}$ are listed in Table 5. Similarly to the conclusion drawn by $R^2$ investigation, the IM $S_{d-T1}$ is the most efficient due to the smallest dispersion. However, the IMs including PGD, PSD, and SED are not efficient as the $\beta_{e|IM}$ values of these IMs are relatively high compared to others.

$$\beta_{e|IM} = \sqrt{\frac{\sum (\ln(e) - (a \cdot \ln(IMs) + \ln(b)))^2}{N - 2}} \quad (3)$$

During the seismic simulation of these eighty-five original records (ORs), the time-history girder displacements at mid-span present residual displacement, as shown in Fig. 9. The absolute values of residual displacement are gathered and linear regression analysis is conducted to explore a potential relationship between residual displacements and IMs. The selected IMs are shown in Fig. 10. The results indicate that, similarly to other $\epsilon$, the residual displacement is sensitive to spectrum IMs with fundamental period of structure. Hence, the $a$ values of $S_{a-T1}$, $S_{v-T1}$, and $S_{d-T1}$ are 0.710, 0.775, and 0.712 respectively, while the $a$ values of other six IMs do not exceed 0.6. However, for all IMs, the sample points and linear regression function do not present strong fitness due to low $R^2$ values. The maximum $R^2$ is only 0.594 for PSA, which indicates that there is no linear relationship between residual displacement and the selected IMs.

4 Pulse parameters analysis of near-fault pulse-type ground motions

4.1 Method of ANRs synthesis

In order to conduct further research of HSRSSB dynamic response under excitation of near-fault pulse-type ground motions, pulse parameters are investigated and analyzed.
in this section. The pulse is regarded as a form of wave. Its parameters, considered as main research objects, are the following: \( A \), amplitude of pulse; \( T \), main pulse period of ground motion; \( N \), number of crests in one period. However, due to lack of near-fault pulse-type ground motions records, Artificial Near-fault Records (ANRs) of ground motions are utilized.

The synthesis procedure of ANRs proposed by Li et.al (2017) can be summarized in the following 3 steps:

1. Ground motion decomposition: the original near-fault pulse-type ground motion records ORs, retrieved from PEER, are decomposed by Butterworth 4th order filter. The low-pass Butterworth filter decomposes the records into two different parts, according to Eq. (4): the high-frequency part (BGR) and the low-frequency pulse part (PTR). The frequencies of BGR and PTR are controlled by the cutoff frequency \( f_c \) calculated by Eq. (5).

![Linear Regression of girder residual displacement at mid-span and IMs](image-url)
\[ v_g(t) = v_{g,BGR}(t) + v_{g,PTR}(t) \]  
(4)

\[ f_c = \frac{1}{\alpha T_P - dt} \]  
(5)

where \( \alpha \) is an empirical coefficient, \( T_P \) is the period for main pulse of time-history velocity, and \( dt \) is the time step.

2. PTR simulation: the main pulse simulation is expressed by Eq. (6), as proposed by Mavroeidis (2003).

\[
v(t) = \begin{cases} 
\frac{A}{2} \left[ 1 + \cos\left(\frac{2\pi f_p}{\gamma}(t - t_0)\right) \right] \cos \left[ 2\pi f_p(t - t_0) + \nu \right], & \text{if} \quad t_0 - \frac{\gamma}{2f_p} \leq t \leq t_0 - \frac{\gamma}{2f_p}, \quad \text{with} \quad \gamma > 1. \\
0, & \text{otherwise}
\end{cases}
\]  
(6)

where \( A \) is the amplitude of artificial time-history velocity, \( f_p \) is the prevailing frequency of the pulse, \( t_0 \) specifies the epoch of the envelope’s peak, \( \gamma \) is a parameter that defines the oscillatory character, and \( \nu \) is the phase of the amplitude-modulated harmonic. The PTR simulation can be regarded as a trial–error procedure. The parameters mentioned above are adjusted until the pseudospectral velocity, spectral velocity, and spectral acceleration of PTR of original ground motion and artificial PTR (APTR) show a good agreement. Detailed simulation procedures can be referred to research proposed by Mavroeidis (2003).

3. Superposition: once the artificial PTR is obtained, the ANRs can be synthesized through combination of BGR and artificial PTR, as given by Eq. (7).

\[ v_{g,ANR}(t) = v_{g,BGR}(t) + v_{g,Artificial,PTR}(t) \]  
(7)

### 4.2 ANRs synthesis and spectrum validation

Six near-fault pulse-type ORs are chosen from the eighty-five earthquake events mentioned before. Their basic parameters are listed in Table 6. These six ORs from different earthquake events are categorized into two types according the pulse type. Hence, the events

| Earthquake event   | Station          | Magnitude | \( R_{rup} \) (km) | \( T_p \) (sec) | Pulses type |
|--------------------|------------------|-----------|--------------------|----------------|-------------|
| E1 "Chi-Chi-Taiwan"| "TCU051"         | 7.62      | 7.64               | 10.381         | F–D         |
| E2 "Imperial Valley-06"| "El Centro Array #4"| 6.53      | 7.05               | 4.788          | F–D         |
| E3 "Northridge-01"| "Sylmar—Converter Sta East"| 6.69      | 5.19               | 3.528          | F–D         |
| E4 "Tabas-Iran"   | "Tabas"          | 7.35      | 2.05               | 6.188          | F–D         |
| E5 "Kocaeli-Turkey"| "Arceilk"        | 7.51      | 13.49              | 7.791          | F–S         |
| E6 "Chi-Chi-Taiwan"| "TCU087"         | 7.62      | 6.98               | 10.395         | F–S         |
including "Chi-Chi-Taiwan", "Imperial Valley-06", "Northridge-01", and "Tabas-Iran" are defined as forward directivity (F–D) pulse-type ORs due to the double-peak or multi-peak reciprocating velocity pulse, while "Kocaeli-Turkey" is fling-step pulse-type ORs with unimodal velocity peak. Based on Table 6 and the ANRs synthesis procedure mentioned above, the empirical parameter $\alpha$ and the synthetic parameters $A$, $f_p$, $t_0$, $\gamma$, and $v$ that are obtained through trial and error process are listed in Table 7.

The ANRs synthesis results are illustrated in Figs. 11, 12, 13, 14, 15 and 16. The diagrams (a) and (b) of these Figures illustrate PTR simulation and time-history velocity simulation comparison, respectively. The diagrams (c) to (f) give the validation reference for response spectrum and pseudo spectrum of velocity and acceleration. For all these earthquake events, the APTRs are consistent with the main pulses of PTRs, and the ANRs obtained by the combination of APTRs and BGRs are in good agreement with ORs, as well. Under the scenario of earthquake E1, the absolute maximum velocity of APTR, 63.7 cm/s, which occurs at 37.9 s, while the corresponding one of PTR is 65.08 cm/s,
occurring at 37.35 s. The error between these maximum velocities is only 2.1%. For other earthquake events, the errors of simulation results do not exceed 5%. Similarly, the spectral acceleration and pseudospectral acceleration of ANRs and ORs are in agreement, as

Fig. 12 ANRs synthesis results of E2, a APTR, b velocity synthesis, c spectral velocity, d spectral acceleration, e pseudospectral velocity, f pseudospectral acceleration

Fig. 13 ANRs synthesis results of E3, a APTR, b velocity synthesis, c spectral velocity, d spectral acceleration, e pseudospectral velocity, f pseudospectral acceleration
shown in Figs. 11d, f to 16d, f. Deviations are noted in diagrams (c) and (e) for earthquake events E1, E3, E4, E5, and E6. The spectral velocity and pseudospectral velocity of ORs and ANRs are consistent when the periods are small. For instance, these two kinds of velocity

Fig. 14 ANRs synthesis results of E4, a APTR, b velocity synthesis, c spectral velocity, d spectral acceleration, e pseudospectral velocity, f pseudospectral acceleration

Fig. 15 ANRs synthesis results of E5, a APTR, b velocity synthesis, c spectral velocity, d spectral acceleration, e pseudospectral velocity, f pseudospectral acceleration
of ANR and OR are in good agreement when the period does not exceed 4 s under the case of E1. This is because the high frequency BGRs of OR and ANR are the same. The errors observed when period is high (low frequency part) are caused by the deviation between PTRs and APTRs. As mentioned above, the ANRs synthesis is mainly aimed at the simulation of the main pulses, thus, the other pulses (except the main one) are directly neglected and replaced by 0. As a conclusion, the errors are inevitable but do not affect the feasibility of the ANRs synthesis.

4.3 Results of pulse parameter analysis

The main pulse parameters researched in this section are: $A$, amplitude of pulse; $T$, main pulse period of ground motion; $N$, number of pulses in one period. The parameters listed in Table 7 are adjusted to obtain the required ANRs for E1 to E6. In order to ensure that the values of $A$ and $T_p$ are within a rational range, the PGV and $T_p$-pulse period of the eighty-five near-fault pulse-type ORs, chosen in Sect. 3.1, are gathered and illustrated in Fig. 17. The maximum PGV is 249.6 cm/s, while for all other earthquake ORs, the PGV does not exceed 152 cm/s.

According to existing realistic data, the $A$ value ranges between 50 and 250 cm/s with 50 cm/s interval, resulting in five different values for all E1 to E6. Regarding the pulse period, it can be noted that nearly 58.8% of all ORs have $T_p$ values that do not exceed 4 s, 25.9% $T_p$ of ORs are in the range of 4–8 s, and only 15.3% ORs have $T_p$ values above 8 s. Due to the maximum and minimum values of $T_p$ being 12.285 s and 0.518 s, respectively, the $T_p$ parameters are set as follows: $3 \times T_s$, $5 \times T_s$, $10 \times T_s$, $15 \times T_s$, $20 \times T_s$, $25 \times T_s$, where $T_s$ is the first-order natural period of HSRSSB modeled in Sect. 2, equal to 0.242 s. The parameter $N$ is only for F–D pulse type ground motions E1 to E4 due to their characteristics.
the F–S pulse type has only unimodal velocity peak, the parameter \( N \) is not investigated for ground motions \( E_5 \) and \( E_6 \). The simultaneous adjustments of \( v \) and \( \gamma \) are utilized to achieve pulse number between 2 and 5. Figure 18 illustrates different pulse numbers for \( E_1 \). Meanwhile, in order to obtain the effect of single parameter on the dynamic response of HSRSSB model, the parameters \( \alpha, f_c, f_p, A \), and \( t_0 \) take the values listed in Table 7.

Figures 19 and 20 show the maximum results of mid-span displacement and pier bending moment, respectively, under different near-fault pulse-type seismic excitation of various parameters. In general, the pier bending moment and girder displacement have positive linear correlation relationship with the pulse amplitude \( A \). Take earthquake ORs \( E_1, E_2, \) and \( E_3 \) for instance, when pulse amplitude increases from 50 to 250 cm/s, the girder displacements increase 30.0%, 12.1%, and 8.9%, respectively, while the pier bending moments increase 33.9%, 13.7%, and 6.1%, respectively. Although there are differences in the values of the dynamic response and the growth rates for these six ORs excitations, the conclusion of linear correlation between \( A \) and dynamic response of HSRSSB still can be drawn. However, under the case of OR \( E_4 \), the absolute maximum girder displacement for pulse amplitude 50 cm/s is approximately equal to the corresponding value for amplitude

![Figure 17](image1.png)

**Fig. 17** ORs distribution based on PGV-\( T_p \)

![Figure 18](image2.png)

**Fig. 18** Number of pulses in one period (\( E_1 \)), a \( N = 2 \), b \( N = 3 \), c \( N = 4 \), d \( N = 5 \)
In order to explain this deviation from the rule for $E_4$, the time-history comparison of velocity, acceleration, and girder displacement between 50 and 250 cm/s cases are illustrated in Fig. 21. When the pulse amplitude $A$ is 50 cm/s, the pulse effect of time-history velocity is insignificant. The value of time-history velocity is controlled by the high frequency BGR, because these high frequency parts are closer to the 1$^{st}$ natural frequencies of the HSRSSB model studied in this paper. They may also lead to severe response of the structure. The comparison diagrams of acceleration and displacement indicate that, although the maximum values of acceleration are similar for both amplitudes, the pulse
effect of 250 cm/s ANR causes a downward movement, which is consistent with the direction of the velocity pulse of the time-history displacement. This leads to opposite signs of the maximum values of the velocity and acceleration shown in Fig. 21a, b. As between 11 and 12 s, the 50 cm/s ANR do not have a negative velocity pulse, the maximum value of girder displacement, 39.05 mm, appears near 11.5 s, while for the scenario of 250 cm/s amplitude, the maximum value appears at 11.8 s, which is −41.73 mm.

The $T_p$ values range between 0.726 s and 6.05 s. It can be noted in Figs. 19b and 20b that for all ORs from $E_1$ to $E_6$, the maximum girder displacement and pier bending moment occur when the pulse period is $3 \times T_s$, meaning 0.726 s. For ORs $E_1$, $E_4$, $E_5$ and $E_6$, the dynamic responses of HSRSSB decrease when the pulse period increases, presenting an exponential function with gradually decreasing rate of change. Typical examples are the curves of the girder displacement of $E_4$ and pier moment of $E_1$. When the pulse period exceeds $10 \times T_s$, the dynamic responses under $E_2$ and $E_3$ ORs excitations increase logarithmically, but the maximum values of dynamic response still appear when $T_p = 0.726s$. Overall, when the pulse period $T_p$ is approaching the first-order natural period $T_s$ of HSRSSB, the dynamic responses under all three kinds of pulse parameters have maximum values within the range of investigation. This is due to the similarity between $T_p$ and $T_s$ which leads to 1st mode vibration of HSRSSB model. As the 1st mode vibration has the largest contribution to the overall response of the simply-supported beam, the parameter analysis results of $T_p$ are shown in the Figs. 19b and 20b.

The investigation of the parameter $N$ of F–D pulse type ground motions (Figs. 19c and 20c) indicates that there is no obvious relationship between the dynamic response of HSRSSB and the number of pulses in one period. The girder displacement and pier bending moment change with odd–even changes of $N$ value. The ORs with the same odd–even feature have approximately maximum response values. However, due to the relatively small difference in the values, it is difficult to draw any meaningful conclusion about the parameter $N$.

For all six ORs, simulation results show that under earthquake $E_2$ and $E_4$, the residual displacement occurs at the mid-span girder. However, the parameter analysis illustrated in Figs. 22 and 23 indicates that the relationship between the residual displacement and all three kinds of pulse parameters is obscure. The residual displacement shows an irregular distribution with the variation of $A$ or $T_p$. For $E_2$, the residual displacement of odd and even $N$ values have opposite signs, while the residual displacements under
E_4 are all negative. Hence no general conclusion can be drawn, and more research is required to establish a specific relationship between N and residual displacement.

Figure 24 demonstrates the effects of these three pulse parameters on other bridge components including girder displacement at the beam end, D_e, and rail displacement at the mid-span, D_r. Moreover, the girder displacement at the mid-span, D_m, is illustrated for comparison. For parameters A and T_p, HSRSSB dynamic response under ORs E_1 and E_5 are chosen for both F–D and F–S type earthquake motion, whilst for N, ORs E_1 and E_2 are selected as examples. The subscript on the diagrams denotes the response results under corresponding OR; for instance, D_e1 is the girder displacement at the beam end under the E_1 seismic excitation. It is noted that under different longitudinal pulse-type ANRs with different BGRs, the maximum values of D_r are basically the same as the values of D_m. The maximum values of D_e are slightly smaller than the D_m, but the difference is within 10%. The changing trends of D_r and D_e caused by variety of the pulse parameters are in consistence with the ones of D_m. These results indicate that although the HSRSSB is modelled in the form of a 4-layer beam in a longitudinal near-fault pulse-type earthquake simulation, due to the stiffness provided by the interlayer components, the longitudinal components can still exhibit similar dynamic responses as an integrated structure.
5 Conclusions

The dynamic responses of HSRSSB under near-fault pulse-type ground motions are investigated from two perspectives, which are IMs correlation analysis and pulse parameters investigation. Eighty-five pulse-type ground motions with fault distance ranging from 0 to 30 km are collected from PEER. They are imposed as longitudinal seismic excitation on an innovative HSRSSB model to obtain the dynamic responses of different bridge components. The innovative model utilized in this paper, called CBM, aims at enhancing the computational efficiency and simplification of modeling procedures. The CBM is based on the FE theory and simulates the HSRSSB as typical 4-layer beam. By boundary condition adjustment of the target spans, the CBM only utilizes four spans to accomplish the modeling of the prototype bridge, which is assumed to be a 49-span simply supported bridge with 10 m pier height. Seismic simulation comparisons with a conventional model verify the CBM’s accuracy of simulation which corresponds to the dynamic response of the middle span of the 49-span prototype bridge. On that basis, four near-fault motions with F–D type pulse and two with F–S type pulse are chosen for pulse parameter investigation. Due to the lack of original records of pulse-type ground motions, artificial near-fault records (ANRs) based on Butterworth filter and pulse simulation method proposed by Mavroeidis (2003) are considered. The pulse parameters investigated in this paper are: $A$, amplitude of pulse; $T_p$, main pulse period of ground motion; $N$, number of pulses in one period. By changing these parameters and exploring the corresponding response of the structure components, preliminary relationships between pulse parameters of near-fault pulse-type ground motion and the dynamic response of HSRSSB are obtained. Conclusions are summarized as follows:

1. The IMs correlation analysis explores sixteen IMs and four kinds of dynamic responses including mid-span displacement of rail and girder, pier bending moment and fixed bearing deformation. Linear regression analysis indicates that the dynamic responses of HSRSSB are sensitive to acceleration-type IMs such as PSA, PGA, $S_a0.2$, and $S_aT_1$ due to relatively high $a$ values. IMs like spectral acceleration, velocity, and displacement at fundamental period of structure have strong linear regression consistency with the dynamic response of HSRSSB. Moreover, $S_dT_1$ has the highest $R^2$ value for all four structure responses and is the most efficient IM due to the smallest $\beta_{\epsilon|IM}$ value.

2. When pulse amplitude $A$ values increase from 100 to 250 cm/s, the mid-span girder displacement and pier bottom bending moment have positive linear relationship with $A$. The maximum increase reaches 30% for the displacement and 33.9% for the pier bending moment. However, when $A$ is 50 cm/s, a part of dynamic responses presented in this paper shows a deviation from the linear relationship. The reason is that the response of HSRSSB is controlled by BGRs due to insignificant pulse-effect with relatively low $A$ value. Moreover, under different BGRs, the dynamic response between ANRs of the same $A$ is quite different, which indicates that the dynamic responses arisen by BGRs are significant.

3. The investigation of period of pulse ($T_p$) under six different BGRs demonstrates that the value of pulse period $T_p$ approaching the first-order natural frequency ($T_s$) of HSRSSB leads to $1^{st}$ mode vibration of target structure. As the $1^{st}$ mode contributes by the largest percentage to the overall response, $T_p$ values activate the maximum dynamic response of HSRSSB. As for variation tendency, in general, when $T_p$ is far from $T_s$, the bridge response attenuates exponentially. Different with pulse amplitude and period of pulse,
the relationship between dynamic response and $N$ number of pulses in one period, is obscure. Due to irregular variation of HSRSSB response with changing of $N$, further research needs to be conducted in future to get potential explicit conclusion of $N$.

4. For residual displacement at mid-span girder, the most sensitive IMs is $S_{v-T1}$, while PSA has the best linear regression consistency. However, the relatively low $R^2$ value indicates that the linear relationship between residual displacement and IMs is not in good agreement. Due to the irregular distribution and variation given by parameter analysis, the relationship between girder residual displacement and pulse parameters is obscure.

5. The maximum displacements at mid-span of rail and girder under longitudinal near-fault pulse-type seismic excitation are similar. The difference between girder displacement at mid-span and at beam-end is within 10%. The variation tendency of these three kinds of displacement under different pulse parameters is consistent. Nevertheless, further investigation is required to prove if this conclusion can be applied for transverse and vertical near-fault pulse-type seismic excitation on HSRSSB.

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**Declarations**

**Conflict of interest** The authors declare that they have no conflict of interest.

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