Vibration suppression of steel truss railway bridge using tuned mass dampers

Rakshith S. Shetty\textsuperscript{1}, Prashanth. M. H\textsuperscript{2}, Channappa. T.M\textsuperscript{3}, Ravikumar. C.M\textsuperscript{4}

\textsuperscript{1} Post graduate student in structural engineering, NITK, Surathkal.
\textsuperscript{2} Assistant Professor, Department of Civil Engineering, NITK, Surathkal.
\textsuperscript{3} Lecturer (Selection Grade), D.R.R. (Govt.) Polytechnic, Davanagere, Karnataka.
\textsuperscript{4} Assistant Professor, Department of studies in Civil Engineering, UBDTCE, Davangere.

channappatm@gmail.com
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ABSTRACT

Structural damages and economic losses caused by natural disaster like earthquakes concerns about structural safety and led to the proposal of various strategies to improve the dynamic performance of bridges. The steel truss railway bridge experiences dynamic vibrations due to train loading and earthquake loading. This paper presents the application of passive control device such as Tuned Mass Dampers (TMD) to steel truss railway bridge, which is used to reduce the vibration of bridge structures. The steel truss railway bridge is subjected to live load such as Heavy Mineral Loading (HMT) i.e. train loading and earthquake loading. The bridge structure is modelled using commercially available finite element software SAP 2000. The response parameters such as displacement, accelerations are observed for steel truss Railway Bridge with and without TMD for HMT loading and earthquake loading. The reduction in the displacement and acceleration are observed for bridge structure with TMD when compared to bridge structure without TMD.

Keyword: Tuned mass dampers, Mass ratio and Steel girder, displacement, acceleration.

1. Introduction

Steel truss bridge have been used in the economically in the span of 100 to 200m. Trussed bridges are economical since the members are primarily subjected to axial forces and open web construction facilitates the use of larger depths with reduction in the self weight. The dynamics of railway bridge involves the response of bridges to the movement of trains and earthquake loading. These vibrations significantly increase the maximum internal stresses of bridges in excess of those originally assumed by the designer. This affects the safety and serviceability of structure. One of method for reducing the vibration of structures in structure is to add an energy dissipative system to the main structure to dissipate a substantial amount of vibration energy of the main structure. The Tuned Mass Damper (TMD) is a secondary vibration system connected to the primary structure at suitable points.

Den Hartog was first to investigate the optimum values of TMD parameters using a two-degree of freedom model in 1950 which have been extensively studied and applied to suppress vibrations of buildings and bridges. TMD is effective in suppressing the single-mode resonant vibration when its frequency is tuned to the modal frequency of the structure. Lin et.al (2005) showed the applicability of passive tuned mass dampers (PTMDs) to suppress train-induced vibration on bridges. A single PTMD system was designed to alter the bridge dynamic characteristics to avoid excessive vibration. Numerical results from simply supported bridges of Taiwan High-Speed Railway (THSR) design proposal subjected to German I.C.E. Japanese S.K.S. and French T.G.V. trains show that the proposed PTMD is a
useful vibration control device in reducing bridge vertical displacements, absolute accelerations, end rotations and train accelerations during resonant speeds. Fahim sadek and Bijan Mohraz (1997) determined the optimal parameters of tuned mass dampers that results in reduction of response to earthquake loading. The proposed method is used to select TMD for several S-DOF and M-DOF systems. The results indicated that the proposed TMD parameters reduces the displacement and acceleration responses significantly up to 50%. Lin et.al (2005) showed the applicability of multiple tuned mass dampers MTMDs to suppress train-induced vibration on bridges. An MTMD with optimally distributed natural frequencies is developed to reduce the four aforementioned dynamic responses of high-speed railway bridges. The maximum vertical acceleration of the bridge was reduced up to 57% for a 2% MTMD mass ratio. Douglas P. Taylor used the TMDs for pedestrian bridges in Las Vegas. Here 6 TMDs were used with a damping of around 20%. It also demonstrated that it is possible to design and construct a TMD that can easily be tuned in the field by simply bolting on additional plate masses, or changing damping and spring characteristics with simple bolt-on elements. In the present paper the response parameters such as displacement, accelerations are observed for steel truss railway bridge with and without TMD for HMT loading and earthquake loading.

2. Model of steel truss railway bridge

![Figure 1: 3-D model of Steel Truss Railway Bridge](image1.png)

The Figure 1 shows the 3-D model of a steel truss railway bridge. The model is done using commercial finite element software SAP 2000. The span of bridge is 78.8m.

![Figure 2: Position of Five TMD in the cross length at mid-span of the bridge](image2.png)

Steel truss bridge consists of bottom lateral bracings, cross girders, portal bracings, stringers, sway bracings, sway girders and verticals. The total weight of the bridge structure.
is 2541.23 kN. Totally five TMD’s are positioned in the cross length at the mid span of the bridge as shown in the Figure 2.

3. Design of the Tuned Mass Damper

![Figure 3: Schematic diagram of the System attached with TMD](image)

**Tuned Mass damper generally consists of mass, spring and damping as shown in Figure 3.** The Tuned Mass Damper has two main roles; firstly to reduce the resonance response of the main structure and secondly, attached dash-pot increases overall damping of the structure by providing additional source of energy dissipation. The M, K, C are the mass, stiffness and damping of the main system respectively, whereas m, k, c are the mass, stiffness and damping of the TMD respectively.

The design of the tuned mass dampers is done using Den Hartogs procedure. The main parameters in design of Tuned mass damper are mass ratio, frequency ratio and damping ratio. From the above parameters, the stiffness of the TMD is tuned, so as to reduce the vibration in the structure.

Den Hartog’s parameters: The formulae given below are used for design of Tuned Mass Damper.

\[
\text{Frequency ratio } f = \frac{1}{1+\mu} \sqrt{\frac{2-\mu}{2}} \quad (1)
\]

\[
\text{Mass ratio } \mu = \frac{m}{M} \quad (2)
\]

\[
\text{Damping ratio: } \xi = \frac{3\mu}{8(1+\mu)} \quad (3)
\]

Mass of the bridge ‘\(M_b\)’ = M = 2, 54,123 kg
(Obtained from the analysis)

Mass ratio ‘\(\mu\)’ = 5 % (assumed)

Mass of the TMD ‘\(M_t\)’= m = 12,706 kg

Five TMDs are incorporated.

Weight of unit TMD = 25.412 kN each

Stiffness of the bridge structure ‘\(K_b\)’ = 167185 kN/m
Damping ratio $\xi = 0.1359$ (13.59 %)

Damping of the TMD = 121.74 kN-m/s

Different earthquakes have different values of frequencies which covers the wide range of frequencies. The stiffness of the TMD is calculated from the frequency of the earthquake. In the present analysis fifteen different earthquakes have been considered. Total Stiffness of the TMDs has been arrived from five earthquakes of higher frequencies.

Stiffness of the TMD ‘$K_t$’ = 15175 kN/m

4. Analysis

Static and Dynamic analysis are performed on the steel truss railway bridge.

4.1 Static analysis

Static analysis is performed to find the dead load and live load behavior of the structure. Dead load is the self weight of structure. Live load analysis is performed for Heavy Mineral loading (HMT).

![Figure 4: Heavy Mineral loading (HMT loading)](image)

4.2 Dynamic analysis

Dynamic characteristic properties of the bridge structure are required to perform the dynamic analysis of the bridge structure. The Modal analysis is used to determine the natural mode shapes and natural frequencies of bridge structure during free vibration. The types of equations which arise from modal analysis are those seen in Eigen systems.

![Figure 5: Time history plot of Uttarkashi earthquake](image)
The Eigen values and Eigen vectors are obtained solving the system equations which represent the natural frequencies and corresponding mode shapes. Time history analysis is also performed for fifteen different earthquakes. The displacement and acceleration at nodes of bridge structure is determined for the time-history function of Uttarkashi earthquake.

5. Results and discussion

5.1 Static analysis

When the live load such as HMT loading is made to move on steel truss bridge, the following were the observations of maximum displacements (shown in Table 1) and maximum acceleration (shown in Table 2) at nodal points from the analysis of the bridge structure with and without TMD.

Table 1: Displacement at the nodes of the bridge structure for the dead load and live load (HMT loading)

<table>
<thead>
<tr>
<th>Node</th>
<th>Maximum Vertical Displacement (m)</th>
<th>Without TMD</th>
<th>With TMD</th>
<th>% Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>52</td>
<td></td>
<td>0.00423</td>
<td>0.00390</td>
<td>7.80</td>
</tr>
<tr>
<td>51</td>
<td></td>
<td>0.00405</td>
<td>0.00369</td>
<td>8.89</td>
</tr>
<tr>
<td>53</td>
<td></td>
<td>0.00389</td>
<td>0.00335</td>
<td>13.88</td>
</tr>
<tr>
<td>63</td>
<td></td>
<td>0.000056</td>
<td>0.000052</td>
<td>7.14</td>
</tr>
<tr>
<td>42</td>
<td></td>
<td>0.00046</td>
<td>0.00039</td>
<td>15.22</td>
</tr>
</tbody>
</table>

* 52 is node at mid-point of the span.
* 51 is node 7m away to the left from the mid-span.
* 53 is node 7m away to the right from the mid-span.
* 63 and 42 are nodes at end of span or support joints

Table 2: Acceleration at the nodes of the bridge structure for the dead load and live load (HMT loading)

<table>
<thead>
<tr>
<th>Node</th>
<th>Maximum Vertical Acceleration (m/s²)</th>
<th>Without TMD</th>
<th>With TMD</th>
<th>% Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>52</td>
<td></td>
<td>5.166</td>
<td>4.612</td>
<td>10.72</td>
</tr>
<tr>
<td>51</td>
<td></td>
<td>0.47</td>
<td>0.39</td>
<td>17.02</td>
</tr>
<tr>
<td>53</td>
<td></td>
<td>0.48</td>
<td>0.41</td>
<td>14.58</td>
</tr>
<tr>
<td>63</td>
<td></td>
<td>3.18</td>
<td>2.817</td>
<td>11.42</td>
</tr>
<tr>
<td>42</td>
<td></td>
<td>2.99</td>
<td>2.717</td>
<td>9.13</td>
</tr>
</tbody>
</table>

The reduction in the displacement and acceleration are observed for bridge structure with TMD when compared with bridge structure without TMD.
5.2 Dynamic analysis

5.2.1 Modal analysis

Modal analysis is usually conducted based on linear Eigen value analysis, which is used to find the natural frequency of steel truss bridge structure and corresponding mode shapes. From the modal analysis, twelve modes of vibration of bridge structure were obtained. Figure 6 and Figure 7 show the first mode and second mode of vibration respectively. The frequency of the first mode is used in calculating the stiffness of the bridge structure. Table 3 shows the values of time period for twelve modes obtained from the analysis for bridge structure with and without TMD.

Table 3: Time period variation

<table>
<thead>
<tr>
<th>Mode No</th>
<th>Time period (seconds)</th>
<th>Without TMD</th>
<th>With TMD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.2472</td>
<td>0.2960</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.1121</td>
<td>0.2649</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.0908</td>
<td>0.2365</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0.0669</td>
<td>0.1798</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>0.0669</td>
<td>0.1530</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>0.0669</td>
<td>0.1300</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>0.0669</td>
<td>0.1118</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>0.0668</td>
<td>0.0908</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>0.0668</td>
<td>0.0669</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>0.0667</td>
<td>0.0669</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>0.0619</td>
<td>0.0669</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>0.0508</td>
<td>0.0669</td>
<td></td>
</tr>
</tbody>
</table>

Figure 6: Mode 1 with T=0.2472 sec

Figure 7: Mode 2 with T=0.1121 sec
The increase in the time period is observed for the bridge structure with TMD as compared to bridge structure without TMD as tabulated in the Table 3 and shown in Figure 8. There is an increase in the time period of about 16% in the first mode i.e. fundamental mode, and for all the other higher modes (upto Six modes) the variation is more than 48%. This indicates the reduced vibration of the bridge structure with TMD when compared to bridge structure without TMD for the same mode number.

**Figure 8:** Comparison of Time period with and without TMD

### 5.2.2 Time history analysis

The Time history analysis is carried out on steel truss railway bridge. The analysis was performed for 15 different time history function. The Uttarkashi earthquake was used in the present analysis.

**Figure 9:** Displacement at node 52 for the bridge structure with TMD

**Figure 10:** Displacement at node 52 for the bridge structure without TMD
The displacement function of node no 52 of bridge structure with TMD and without TMD was observed when subjected to Time history plot of Uttarkashi earthquake as shown in Figure 9 and Figure 10 respectively.

The maximum displacement (shown in Table 4) and maximum acceleration (shown in Table 5) at nodes of the bridge structure with TMD and without TMD for time-history plot of Uttarkashi earthquake was observed from the analysis.

**Table 4: Displacement at the nodes of the bridge structure for the Earthquake loading**

<table>
<thead>
<tr>
<th>Node</th>
<th>Maximum Vertical Displacement (m)</th>
<th>Without TMD</th>
<th>With TMD</th>
<th>% Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>52</td>
<td>0.00416</td>
<td>0.00357</td>
<td>14.18</td>
<td></td>
</tr>
<tr>
<td>51</td>
<td>0.00387</td>
<td>0.00313</td>
<td>19.12</td>
<td></td>
</tr>
<tr>
<td>53</td>
<td>0.00384</td>
<td>0.00302</td>
<td>21.35</td>
<td></td>
</tr>
<tr>
<td>63</td>
<td>0.000005</td>
<td>0.000004</td>
<td>20.00</td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>0.0000019</td>
<td>0.0000015</td>
<td>21.05</td>
<td></td>
</tr>
</tbody>
</table>

**Table 5: Acceleration at the nodes of the bridge structure for the Earthquake loading**

<table>
<thead>
<tr>
<th>Node</th>
<th>Maximum Vertical Acceleration (m/s²)</th>
<th>Without TMD</th>
<th>With TMD</th>
<th>% Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>52</td>
<td>3.656</td>
<td>2.912</td>
<td>20.35</td>
<td></td>
</tr>
<tr>
<td>51</td>
<td>3.448</td>
<td>2.921</td>
<td>15.28</td>
<td></td>
</tr>
<tr>
<td>53</td>
<td>3.341</td>
<td>2.659</td>
<td>20.41</td>
<td></td>
</tr>
<tr>
<td>63</td>
<td>0.00145</td>
<td>0.00118</td>
<td>18.62</td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>0.000846</td>
<td>0.000538</td>
<td>36.41</td>
<td></td>
</tr>
</tbody>
</table>

* 52 is node at mid-point of the span.
* 51 is node 7m away to the left from the mid-span.
* 53 is node 7m away to the right from the mid-span.
* 63 and 42 are nodes at end of span or support joints

5. Conclusion

From the analysis of the results, the following conclusions can be drawn

1. Bridge structure with TMD is more effective in the reduction of vibration from the response parameter (displacement and acceleration) observed for earthquake loads when compared to structure without TMD.

2. Bridge structure with TMD has lower level of reducing the vibration from the response parameter (displacement and acceleration) for moving load than structure without TMD.

3. TMD are more useful in the control of vibrations due to earthquake loading than due to moving load.
4. The time period is more than 16% in the first mode for the steel truss bridge with TMD, when compared to the steel truss bridge without TMD. For all the other higher modes of vibration, the time period is larger than 48% when compared to steel truss bridge without TMD. This indicates the reduced vibration of the bridge structure with TMD when compared to bridge structure without TMD for the same mode number.

6. References


