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Research Paper

Dynamic response analysis of truss bridges under the effect of moving vehicles

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A B S T R A C T

With the characteristics of heavy and concentrated loads, the influence of moving loads on the dynamic response of the bridges is significant. Therefore, in this paper, the dynamic response of a large-scale truss bridge is studied to consider the effect of the various parameters of moving loads. The considered main parameters consist of moving mass, moving velocity, and type of moving loads. The nonlinear dynamics of the bridge based on time history analysis are obtained using the Wilson-θ method. four time history – based dynamic analysis method including modal superposition in frequency domain, modal superposition in time domain; direct time integration, and direct solution in the frequency domain are employed to analysis the obtained results. To compare the effectiveness of the aforementioned method. A large-scale railway truss bridge is employed for dynamic response analysis. The obtained results give more insight into the nature of the problem and help to determine the significant parameters of moving load affecting the bridge response.

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1 Introduction

The effect of moving loads of trucks, trains, or other types of transport vehicles often causes complex vibrations for bridges. Depending on the velocity of movement, the changes in the position of the loads over time, forced vibrations, resonance vibration, and fatigue failure can occur and exert adverse effects on bridges. These effects can cause unsafety for people and vehicles on the bridge as well as affect the life of the structure. The problem of analyzing the impact of moving

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loads on the bridges is challenging because it depends on many random parameters such as the types of transport vehicles, the characteristics of the structure, the moving speed of the vehicles, and so on [1-3].

Previously, the linear elastic analysis method or the vibration spectrum analysis method was employed to analyze the structural dynamic response. However, while the first method applies too many quantitative requirements, the last one does not consider power dissipation. This may reduce the accuracy of obtained results. To deal with this shortcoming, over the last decades, many researchers have proposed methods based on real-time data (time history method) to analyze the dynamic response of the structure. Some of the most popular methods are the Newmark method [4] , Wilson-θ method [5], Hiber-Hughes-Taylor (HHT) method [6], and so on. For example, Bamer et al [7] analyzed a two-dimensional frame structure subjected to four different excitation functions using Wilson-θ method. Mohammadzadeh et al [8] proposed solutions to the traditional Wilson-θ method. The authors found that the proposed approach not only provides a high degree of accuracy but also achieved controllable amplitude decay. Ozkul [9] presented an approach to analyze the dynamic behaviors of shells by utilizing the Wilson-θ method. Mohseni et al [10] applied the HHT method for dynamic response analysis of a skewed bridge caused by moving loads. Lui et al [11] used the Runge-Kutta method when analyzing the dynamics response of a semi-rigid frame structure. Pasetto et al [12] proposed the Waveform Relaxation Newmark (WRN_β) method to overcome the disadvantages of Newmark methods to analyse the dynamic behavior of 2-dimensional flat plates.

However, most aforementioned authors only applied the time history method to solve dynamic analysis problems of structures in the laboratory or small structures. Therefore, in this paper, we propose using the most effective method based on the time history method, termed, the Wilson-θ method for the dynamic response analysis of a real large-scale steel truss bridge.

In addition, in this research, four time history – based dynamic analysis method including modal superposition in frequency domain, modal superposition in time domain; direct time integration, and direct solution in the frequency domain are employed to analyze the obtained results.

2 Methodology

The basic assumption of the Wilson-θmethod is that the acceleration of the structure changes linearly from time t to time $t' = t + \theta.\Delta t$; θ $(\theta \geq 1)$ is determined based on optimizing the stability and accuracy of the calculation results. From the time interval $t$to $t + \theta.\Delta t$, we have:

$$a_{t+\sigma} = a_t + \left(a_{t'} - a_t\right).\frac{\sigma}{\Delta t} \tag{1}$$

$$v_{t+\sigma} = v_t + a_t.\sigma + \left(a_{t'} - a_t\right).\frac{\sigma^2}{2.\Delta t} \tag{2}$$

$$d_{t+\sigma} = d_t + \sigma.v_t + \frac{1}{2}.\sigma^2.a_t + \left(a_{t'} - a_t\right).\frac{\sigma^3}{6.\Delta t} \tag{3}$$

with $0 \leq \sigma \leq \theta.\Delta t$; at time $t + \Delta t$, we have:

$$v_{t+\Delta t} = v_t + \left(a_{t+\Delta t} + a_t\right).\frac{\Delta t}{2} \tag{4}$$

$$d_{t+\Delta t} = d_t + \Delta t.v_t + \left(a_{t+\Delta t} + 2a_t\right).\frac{\Delta t^2}{6} \tag{5}$$

Substituting Eqs. (1), (2) and (3)into the equations of dynamic equilibrium with $\sigma = \theta.\Delta t$, we have:

$$M.a_{t'} + C.v_{t'} + K.d_{t'} = P._{t'} \tag{6}$$

Where M, C, K, and P are the mass, stiffness, and damping matrices, forces, respectively. In this paper, we assume that the system is linear, in which M, C, and K are constant.

Solving Eq.(6)with a single unknown $a_{t'}$, then substituting Eq.(4)and(5), we get the values of displacement, velocity, and acceleration at time t'.

3 Dynamic analysis of a steel truss bridge

3.1 Description: Chuong Duong bridge

Chuong Duong Bridge (Fig.1) is a steel truss bridge built in 1985. The bridge connects Hoan Kiem district with Long Bien district, Hanoi capital (Vietnam), consisting of 11 truss spans. The length of each span is almost equal (90m). In this paper, we focus on considering the dynamic response of 11^{th} span (see Fig.2) under the effect of moving loads.

Fig.1 – Chuong Duong bridge (Vietnam)

Fig.2 – Thegeneral view of 11^{th} span

3.2 Finite Element Model (FEM)

The FEM of 11^{th} span of the bridge (**Fig.** 3) is build by using Matlab.

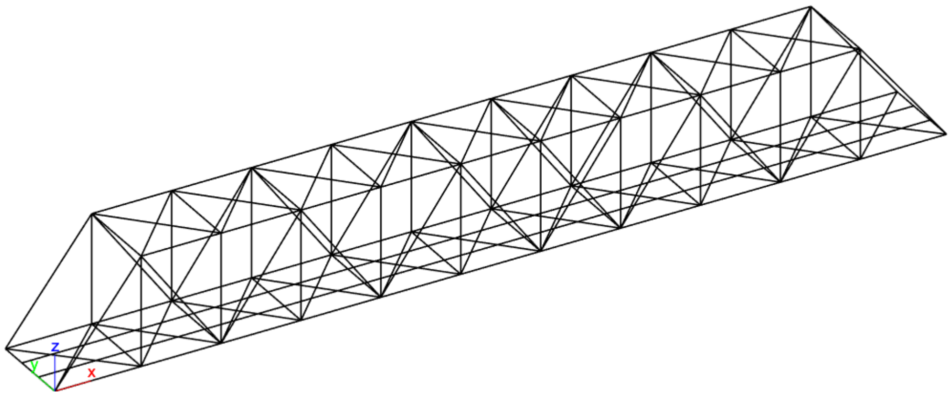

Fig. 3 – The FEM of 11th span of the bridge

Some information about the model:

The number of nodes: the FEM includes69 nodes, in which each node contains 6 degrees of freedom (DOF) corresponding to translational and rotational displacements in the X, Y, and Z axes.

The number of elements: 192 elements are used, including stringers, floor beam, lower chord, upper chord, vertical, diagonal, portal bracing, lower lateral bracing, upper lateral bracing, and so forth.

Boundary conditions: The span are put on rocker and pin bearings. The first bearing permits translation and rotation in one direction, whereas the second one only allows rotational movement.

The input parameters of the material such as elastic modulus (E), density (w), Poisson's ratio (υ) as well as the section (area, moment of inertia) are referenced from the as-built records. The material properties of the truss members are depicted in Table 1.

Table 1 - Material properties of truss members

Components	Value	Unit
Young's modulus	2×10^{11}	Pa
Volumetric mass density	7850	Kg/m3
Poisson's ratio	0.3	/

The geometric properties of the truss members are shown in **Table 2** as follows:

Table 2 - Geometric properties of truss members.

Components	Area (A)	Moment of inertia (I_z)	Moment of inertia (I_y)
	(m^2)	($kg.m^2$)	($kg.m^2$)
Other components (lower chord, upper chord, vertical, diagonal, and so on)	0.067	0.01368	0.02194
Upper lateral bracing	0.008	0.00038	0.00071
Lower lateral bracing	0.0081	0.00036	0.00065

3.3 Dynamic analysis

To analyze the dynamic responses of the structure, the Wilson-θ method is employed. Three moving loads are surveyed to evaluate the influence of moving mass, moving velocity, and type of moving loads (see Table3 and Table 4).

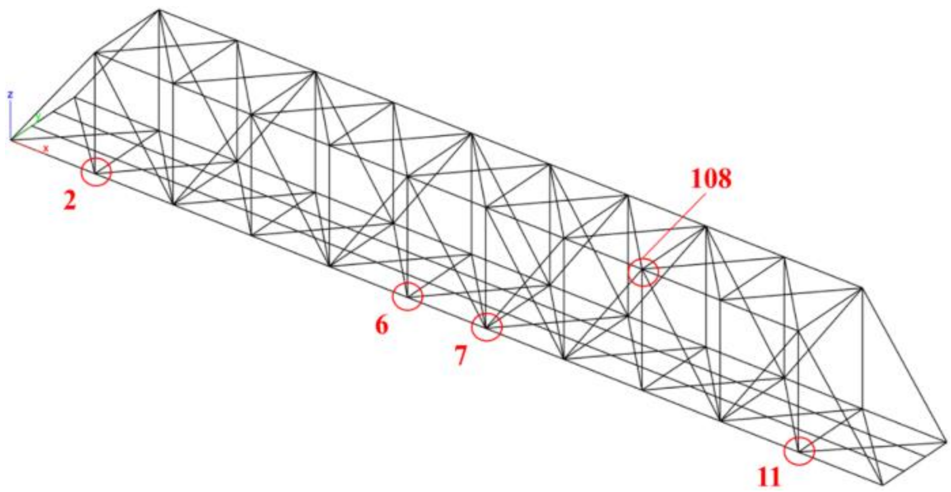

Fig.4 – The position of the nodes determines dynamic displacements following the z −axis

Table3 - Specification of moving loads.

Specification
Axle load: $P1$
Axle load: P2
Axle load: P3
Axis distance $P1 - P2$: $a1$
Axis distance $P2 - P3$: $a2$
Time step Δt: $0.001s$

Table 4 - Types of typical load.

No.	$P1$ (ton)	$a1$ (m)	$P2$ (ton)	$a2$ (m)	$P3$ (ton)	$\sum P$ (ton)
I	4.76	3.70	7.62	1.350	7.62	20.00
II	5.40	3.80	10.30	1.385	10.30	26.00
III	7.00	3.80	11.50	1.85	11.50	30.00

The dynamic analysis of the bridge under the load of three moving loads with the average speed of 40, 50, and 60 km/h are conducted. Four methods including, modal superposition in frequecy domain, modal superposition in time domain, direct solution in the frequency domain, and direct time integration are employed. The results are shown in Fig.5-Fig. 11 and Table 5- Table 8.

Modal superposition in frequency domain

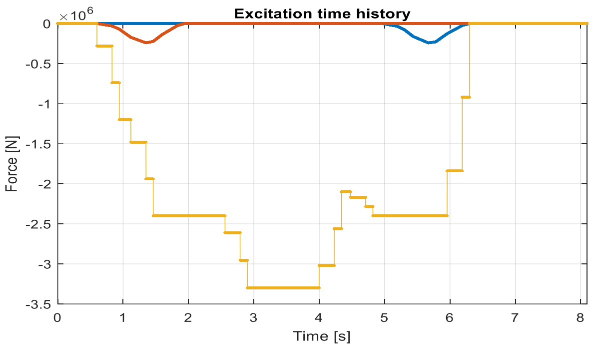

Fig.5 – Excitation time history

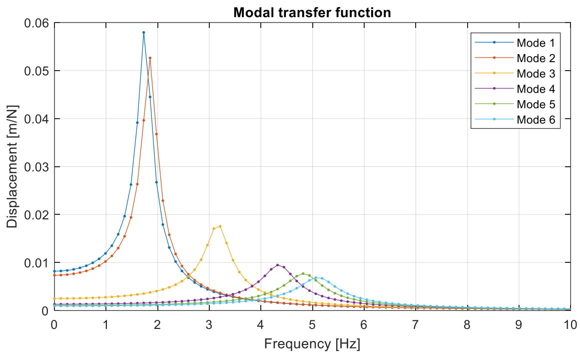

Fig.6 – Modal transfer function

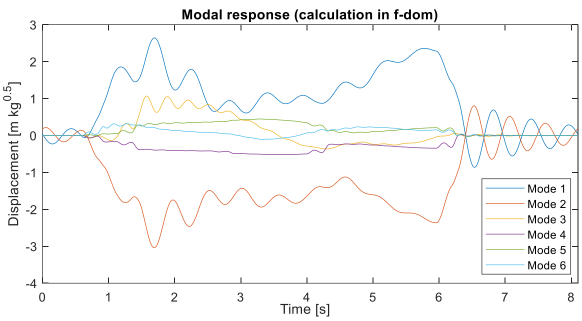

Fig.7 – Modal response frequency domain

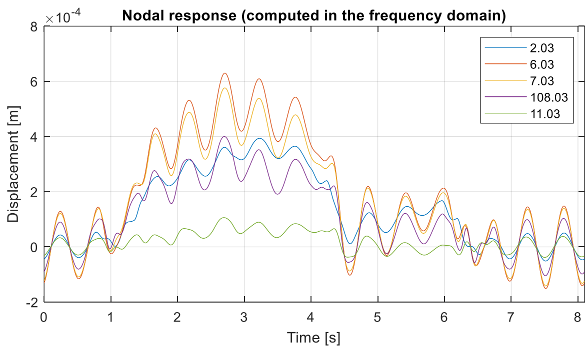

Fig.8 – Dynamic displacement

Table 5 - Dynamic displacement according to modal superposition in frequency domain

Velocity		40 km/h			50 km/h			60 km/h		
Loads		I	II	III	I	II	III	I	II	III
Node (m)	2	0.248	0.324	0.372	0.259	0.340	0.386	0.270	0.354	0.394
	6	0.381	0.499	0.565	0.426	0.565	0.635	0.433	0.574	0.630
	7	0.341	0.446	0.507	0.385	0.511	0.574	0.396	0.526	0.576
	108	0.230	0.302	0.345	0.263	0.351	0.394	0.276	0.367	0.400
	11	0.058	0.077	0.088	0.070	0.094	0.105	0.074	0.099	0.106

Modal superposition in time domain

From obtained results of Fig.5 - Fig.12 and Table 5 - Table 8, we can see that the obtained results from the four methods are almost the same. Specfficially, the displacement at the mid-span (node 6) is the largest in all cases. With the same speed, the maximum displacement along the vertical axis (z-axis) is proportional to the load of the vehicles. In the case of the 3rd moving load, the displacement of the structure when the vehicle moves at a speed of 50km/h is greater than the speed of 60km/h. This demonstrates that the speed of the moving load is not completely proportional to the displacement of the structure.

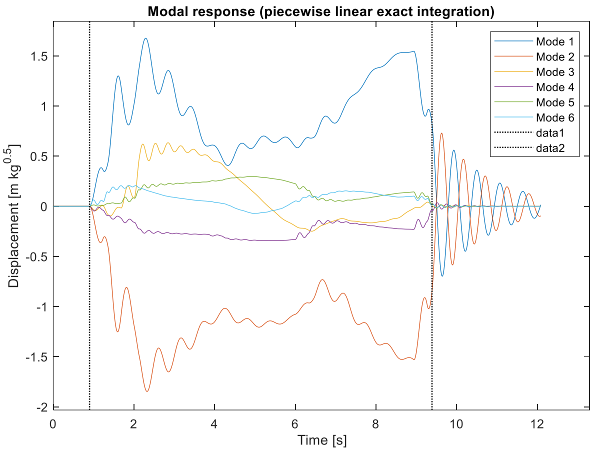

Fig.9 – Modal response time domain

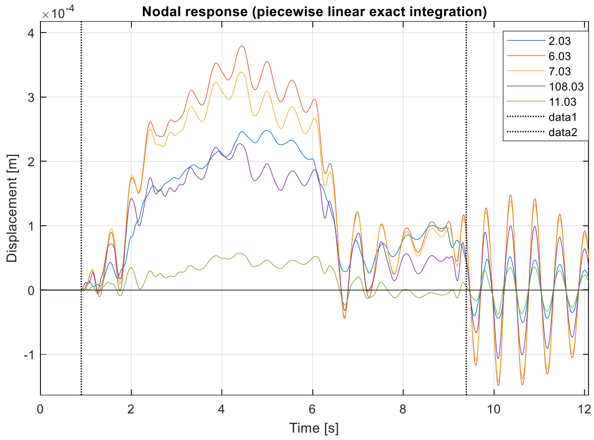

Fig.10 – Dynamic displacement

Data 1 and data 2 in the Fig.9- Fig.10 present the times when moving vehicle enters and move out of the bridge.

Direct time integration

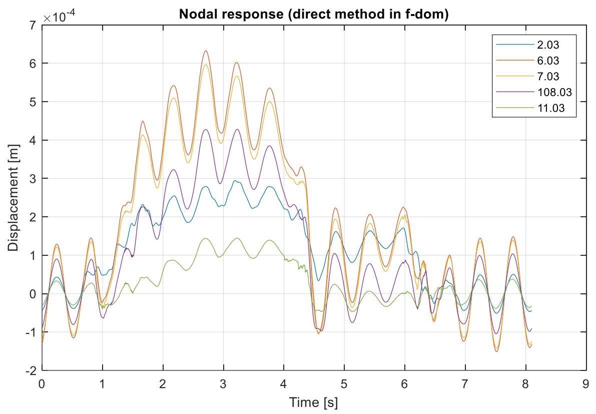

Fig. 11 – Dynamic displacement

Direct solution in the frequency domain

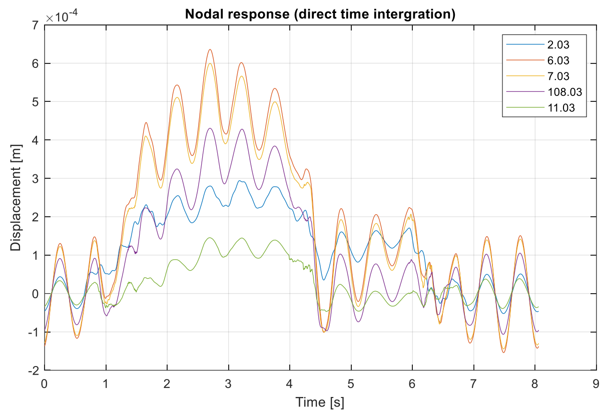

Fig.12 – Dynamic displacement

Table 6 – Dynamic displacement according to modal superposition in time domain method

Velocity		40 km/h			50 km/h			60 km/h		
Loads		I	II	III	I	II	III	I	II	III
Node (m)	2	0.248	0.324	0.372	0.261	0.342	0.389	0.271	0.356	0.399
	6	0.380	0.497	0.564	0.435	0.578	0.648	0.450	0.600	0.660
	7	0.339	0.444	0.505	0.393	0.523	0.586	0.412	0.551	0.604
	108	0.228	0.299	0.343	0.269	0.359	0.402	0.288	0.385	0.420
	11	0.058	0.076	0.087	0.072	0.097	0.108	0.078	0.105	0.113

The reason can be explained based on the resonance as well as the position of the axial loads during the moving process of vehicles. For example, although the vehicles move with lower speed, they can cause a greater displacement for the structure if the axial puts on the mid-span and the resonance phenomenon occurs.

Table 7 – Dynamic displacement according to direct time integration method

Velocity		**40 km/h**			**50 km/h**			**60 km/h**		
Loads		**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**
Node (m)	2	0.187	0.243	0.278	0.200	0.262	0.294	0.205	0.269	0.295
	6	0.380	0.497	0.561	0.425	0.563	0.634	0.434	0.576	0.633
	7	0.357	0.467	0.530	0.400	0.531	0.598	0.410	0.543	0.596
	108	0.264	0.346	0.396	0.292	0.388	0.441	0.294	0.390	0.428
	11	0.090	0.118	0.137	0.101	0.134	0.152	0.099	0.131	0.145

Table 8 – Dynamic displacement according to direct solution in the frequency domain method

Velocity		**40 km/h**			**50 km/h**			**60 km/h**		
Loads		**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**
Node (m)	2	0.188	0.244	0.279	0.198	0.261	0.293	0.205	0.269	0.294
	6	0.382	0.500	0.563	0.423	0.561	0.633	0.438	0.582	0.636
	7	0.358	0.469	0.531	0.398	0.528	0.597	0.413	0.549	0.600
	108	0.265	0.348	0.397	0.291	0.386	0.439	0.297	0.394	0.431
	11	0.090	0.118	0.137	0.100	0.133	0.151	0.100	0.133	0.145

Table 9 shows that modal superposition in the frequency domain and modal superposition in the time domain require lower time to finish the dynamic analysis process of the considered structures than direct time integration and direct solution in the frequency domain.

Table 9 – Computational time.

Velocity	**40 km/h**			**50 km/h**			**60 km/h**		
Loads	**I**	**II**	**III**	**I**	**II**	**III**	**I**	**II**	**III**
Modal superposition in frequency domain	0.42	0.30	0.31	0.30	0.27	0.31	0.24	0.22	0.21
Modal superposition in time domain	0.41	0.37	0.41	0.36	0.32	0.30	0.26	0.27	0.27
Direct time integration	13.29	13.37	13.36	10.72	10.58	10.84	8.96	8.86	8.88
Direct solution in the frequency domain	13.24	13.249	13.29	10.76	10.78	10.65	8.96	8.87	8.93

4 Conclusions

This paper investigates the dynamic response of a bridge under the effect of moving loads. The considered main parameters consist of moving mass, moving velocity, and type of moving loads. To analyze the structural dynamic response, the time history method (Wilson-θ method) is employed. From the obtained results, some main conclusions are drawn:

The displacement along the vertical axis is proportional to the load of the vehicles.

The obtained results from the four aforementioned methods are almost the same. This demonstrates the reliability of the used methods.

Modal superposition in the frequency domain and modal superposition in the time domain outperforms direct time integration and direct solution in the frequency domain in terms of reducing the computational time.

The displacement at the mid-span is the largest in all cases.

The displacement of the bridge is not completely dependent on the speed of the moving load. In other words, the bridge appears maximum displacement when the resonance phenomenon occurs.

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