EFFICACY OF TUNED MASS DAMPER FOR SEISMIC RESPONSE REDUCTION ON STEEL BRIDGES

A DISSERTATION

Submitted in partial fulfillment of the

requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARHTHQUAKE ENGINEERING

(With specialization in Structural Dynamics)

By

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CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation entitled, **"EFFICACY OF TUNED MASS DAMPER FOR SEISMIC RESPONSE REDUCTION ON STEEL BRIDGES"**, in the partial fulfillment of the requirements for the award of the degree of **MASTER OF TECHNOLOGY** in **EARTHQUAKE ENGINEERING,** with specialization in **STRUCTURAL DYNAMICS**, submitted in the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out for a period from July 2010 to June 2011 under the supervision of **Prof. Ashok Kumar**, Department of Earthquake Engineering, IIT Roorkee and **Prof. S. K. Thakkar**, Ex-faculty, Department of Earthquake Engineering, IIT Roorkee.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree or diploma of this institute or any other university/institute.

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Abstract

Use of energy damping systems is often questioned in earthquake resistant design of structures. Its applications are still limited as it has not attained full-recognition. One such system is tuned mass damper system which has proved its efficiency against wind forces. But its efficacy against seismic forces is yet to be justified. Present study is to check the efficacy of tuned mass damper system under seismic forces for steel truss bridges. Efficiency is checked for vertical displacement of mid-span of bridge under vertical ground motions. Steel truss bridges of different spans viz. 45.6 m, 61.0 m and 76.8 m are taken for study. Different time-history records of Uttarakhand region are taken for seismic analysis. An average response spectrum is developed using response spectrum of these time-histories. Again, spectrum based time-histories are generated using this response spectrum. Initially single tuned mass damper system is used to evaluate the effect of its different parameters on vertical displacement of bridge. Thereafter, average response reduction is calculated for all time-histories. Further, 3 TMD systems with different frequency attributes are used for response reduction. Change in average response reduction is observed with shift in frequency band as well as change in frequency band-width of 3 TMD system. Further, 5 TMD system is used, for all bridge models , with different frequency range as well as minor change in one frequency of TMD. Substantial reduction in response is observed when frequencies of TMD system is tuned with the excitation frequencies.

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Chapter 1

Introduction

1.1 Preamble

Rail transport is a means of conveyance of passengers and goods by way of wheeled vehicles running on rail tracks. In contrast to road transport, where vehicles merely run on a prepared surface, rail vehicles are also directionally guided by the tracks they run on.

Railway transport is considered as one of the best transport system among all. For transportation of heavy population and goods, it is found to be most efficient and cost-effective means of mechanized transport. It is also faster than other means on cost-value basis. It is part of the logistics chain, which facilitates national trade and economic growth in most countries.

Indian Railways is the central government-owned railway company of India, which owns and operates most of the country's rail transport. It has more than 64,215 kilometers of track and 7,083 stations. It has the world's fourth largest railway network after those of the United States, Russia and China. The railways traverse the length and breadth of the country and carry over 25 million passengers and 2.5 million tons of freight daily *[4]*.

Railway bridges hold high importance in this large and persistent railway network. It can cut short the travelling distance by large amount. Also at terrains where grounded railway cannot be established, railway bridges may be constructed.

Earthquake can cause a detrimental effect on the bridge thus causing high damage or even collapse of bridge under severe stresses. With quite a number of bridge failures under earthquake, and bridge being a very important structure both strategically and economically, it becomes necessary to design a bridge for earthquake forces also. Normally bridge girders are not affected by horizontal earthquake motions. Vertical vibrations in girders assume importance under vertical earthquake motion. Such effects are important for moderate to long spans. In this thesis, methods of reducing vertical vibration of girder under vertical ground motion are studied for simply supported truss bridges.

1.2 Objective of the study

One of the methods of safe design of important structures to resist earthquake forces is to design it fully elastic on strength basis. But this method of design will render huge increase in the cost of structure. One of the other methods to cater for such vibrations under earthquake is the use of mechanical damping systems. Damping system is a dynamic vibration energy absorbing system. Two types of energy damping mechanisms viz. active and passive energy damping mechanisms can be used for structures. But as most of the railway bridges being located in remote areas, active damping system is generally not useful as they require power for its operation. This leads to an option of using passive energy damping mechanism.

One of such passive mechanisms is Tuned Mass Damper (TMD) system. This methodology may be effectively used for reducing the vertical vibration response of the bridge. This study attempts to study the efficacy of Tuned Mass Damper system for reducing vertical response of steel truss railway bridges under vertical vibrations due to earthquakes.

1.3 Scope of the study

Now most of the railway steel bridges are either girder or truss bridges. Girder bridges are generally used in shorter spans. So they are not much affected by horizontal motion. Also vertical motion can affect them only when their spans are bigger. So large span truss bridges, which are more affected by vertical vibrations, are taken for study.

In this study, three railway steel truss bridges with clear span of 45.7 m, 61.0 m and 75.6 m which are designed for 25 Tonne loading are taken. All the bridges are modeled as single span with simply-supported end conditions. The aim of study is to have reduction of response of bridge using tuned mass damper system with single TMD, 3 TMDs and 5 TMDs. Various earthquake time-histories recorded at different stations of Uttarakhand region is taken and applied as an earthquake loading input function. Design of TMD is known to be hugely dependent on exciting frequencies and therefore it is dependent upon region.

1.4 Methodolgy adopted for the study

Fifteen earthquake time-histories (recorded) from different sites of Uttarakhand are collected for the study. Then, response spectrum is generated from each time-history. From these spectra an average normalized response spectrum is calculated by taking their arithmetic average. Thereafter, four spectrum compatible time-histories are generated from this spectrum. Thus, total of nineteen time-histories are used as input loading functions for the analysis. These time-histories are then applied to mathematical models of bridge of span 45.7 m, 61 m and 75.6 m. Linear direct time-history analysis is then performed for all bridge models under all time-histories with and without TMD. Single TMD system, three TMD system and five TMD system with variation in frequency are used. Thereafter, results are compared for all the cases to draw the conclusion.

Initially, single tuned mass damper system is applied to the bridge. This is to have a preliminary idea of behaviour of response of bridge with respect to various tuned mass damper parameters viz. stiffness, mass and damping of TMD. Further, multiple tuned mass damper system is applied by suspending three tuned mass damper system and finally five tuned mass damper system to the bridge structure. All tuned mass damper systems are suspended vertically, at the mid-span of bridge along its transverse axis as shown in fig. 3.4, also maintaining symmetry about its longitudinal axis. 2D plane frame analysis has been performed. Plane of analysis is contained by vertical and longitudinal axis of bridge. Linear Direct Integration time-history analysis (Hughes-Hilbert-Taylor) method is used for the study. SAP Computers and Structures version 14.2.2 software has been used for the analysis.

Response of the structure is measured in terms of *vertical displacement at the mid-span* of bridge as first mode of simply supported beam (bridge) gives maximum ordinate at mid-span. Also as first mode contributes maximum to the response of bridge, it is obvious to have reduction of displacement at its maximum ordinate i.e. at mid-span of bridge. Now as per working of tuned mass damper system, it should be suspended at the point where reduction in response is sought. Hence all tuned mass damper system used in the study are suspended at mid-span only. So this study is restricted to reduce the vibration response under odd modes only as even mode ordinates at mid-span will be zero under simply supported conditions. Reduction in response of bridge leads to reduction in stresses in bridge and hence damage as a whole.

The following chart gives the idea about the methodolgy adopted for this study.

1.5 Work flow for the study

Collection of time histories of different regions of Uttarakhand

Generation of response spectrum from every time history

Development of average response spectra

Generation of spectrum compatible time histories

Mathematical modeling of steel railway truss bridge and TMD system

Finding vertical displacement response at mid-span of a each bridge model for

- Spectrum compatible time histories without TMD
- Original time histories without TMD
- Spectrum compatible time histories with different TMD systems
- Original time histories with different TMD systems

Comparison of results

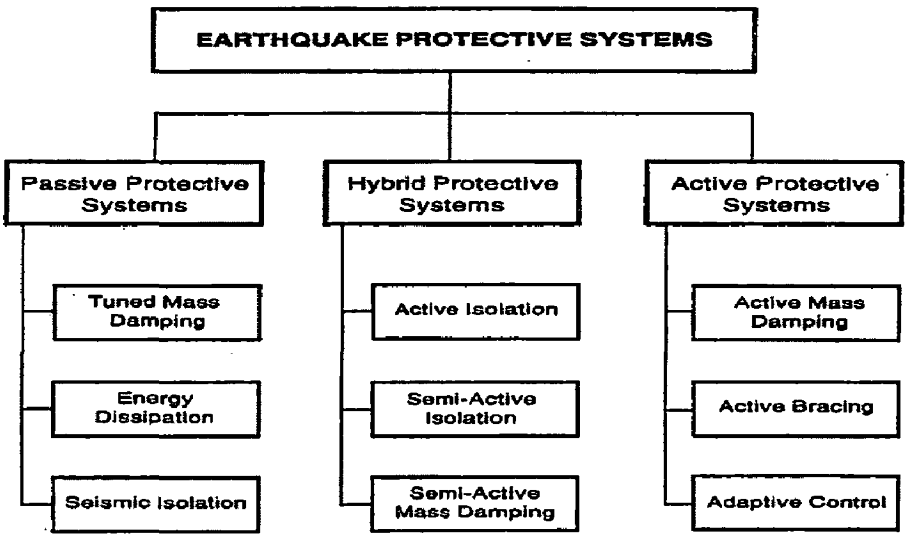
Conclusion

Chapter 2

Tuned Mass Damper System

2.1 General

It has been widely accepted that under dynamic behaviour, design for strength alone does not confirm for controlled response and safety of structural system. In fact the requirements of strength and safety can be conflicting *[2]*. That gave an urge to find alternate ways of improving the resistance of structures with desirable dynamic properties. Many systems were evolved to control the response of the structure under seismic loadings and still continue to develop till date. Such systems include modification in rigidities, masses, damping or shape by providing passive or active counter forces. Some of them are used successfully in past and some methods provides opportunity to extend its applications and improve its efficiency. Flow chart below shows classification of seismic protection systems *[5]*.



2.2 Basis of idea development

There can be two basic situations when one frequency (very close band) dominates vibration response and produce large seismic response.

1. When the excitation frequency is close to fundamental frequency.

2. When there is strong excitation amplitude in a frequency band.

In the first case, a broadband excitation to the system can still produce a response dominated by one mode. In the second case, the modes of the structure (i.e. natural frequencies) may or may not be relevant; the excitation input frequency shows up directly in the response. In each of these situations, a tuned vibration suppression device can be effective. A damper is preferred to reduce resonant response, and an absorber without damping can be used to counter narrowband forced response. The resonant damper is referred as tuned mass damper (TMD) and the undamped absorber as tuned mass absorber or tuned vibration absorber (TVA). Most often, in the structure, the first vibration frequency and mode of the primary system plays a dominant role in the dynamic response. This makes TMD to be an ideal system for reducing seismic response.

2.3 Basic definition of TMD system

TMD is basically a dynamic vibration absorber. It is a device consisting of a mass (secondary mass in a system), a spring, and a damper attached to a primary system as shown in fig1.1. It is used to reduce the dynamic response of the structure. Basically it is an energy sink wherein excess kinetic energy build-up in the structure is transferred to the secondary mass. The frequency of the damper is tuned to a particular structural frequency so that when that frequency is excited, the damper will resonate out of phase with the structural motion. Energy will get dissipated by the damper inertia force acting on the structure.

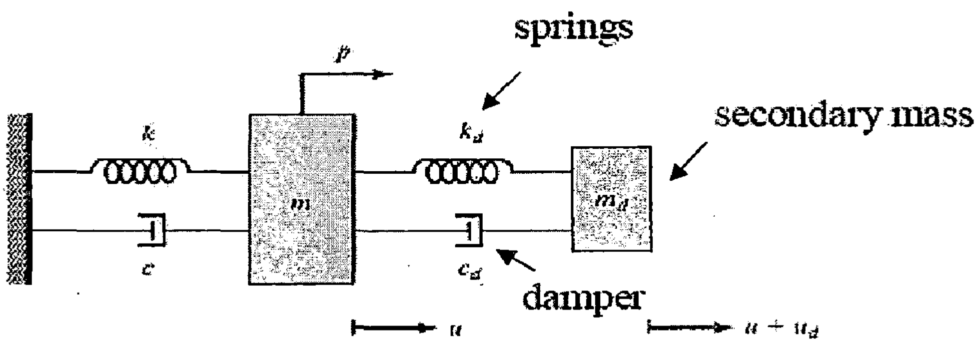


Figure 2.1: Schematic diagram of a SDOF – TMD system

Notations:

d = tuned mass damper; w = natural frequency of the structure; w_d= natural frequency of tuned mass damper; c = coefficient of viscous damping of the structure; c_d = damping of tuned mass damper; k= stiffness in system; k_d = stiffness in tuned mass damper; p = force in m; u= relative displacement in m; u_d = relative displacement at m_d; $\overline{m} = m_d/m$;

2.4 Advantages of TMD system

- It is a passive damping system. So it does not require any power to operate. So they can be easily installed in new as well as in retrofitting of existing building. They can be applied at remote places without power provision and can also prove best under power failure during earthquakes. Thus it can be used as standby to active systems in case of failure of active systems during power failure.

- It does not affect vertical and horizontal load paths.

- The tuned device does not impact the static strength or stiffness of the base structure. i.e. TMD can be installed into the structure with less interference compared to other passive energy dissipation devices.

- Some TMD devices are such inherently compact and modular so as to have a simple interface to the base structure.

- The top floors with appropriate stiffness and damping can be effectively utilised as TMD system.

2.5 Working Principle of TMD system

Tuned mass dampers reduce violent vibrations caused by harmonic vibration. The presence of a tuned damper allows the inertia of a heavy(main) mass to be balanced by a comparatively light-weight structural component, such as a heavy steel block placed in such a way that the tuned mass block moves in one direction as the structure moves in the other, thus damping the structure's oscillation. The counterweight may be mounted using massive spring coils and hydraulic dampers. If the axis of the vibration is fundamentally horizontal or torsional, leaf springs and pendulum-mounted weights are employed. Tuned mass dampers are engineered or tuned to specifically counter harmful frequencies of oscillation or vibration.

2.6 Mathematical equation of TMD system

TMD consists of a secondary mass attached to structure through spring in parallel with the damper as shown in fig. 2.1. Secondary mass moves relative to the basic structure. TMD gets excited under vibrations which cause transfer of kinetic energy from the structure to TMD which then gets absorbed by damping system of TMD. During this process the mass of TMD is under large displacement. The fundamental theory of translational tuned mass damper can be explained as follows *[6]*.

Consider two-mass system as shown in fig. 1.1. System is taken as single degree od freedom system subjected to sinusoidal (harmonic) loading with a secondary mass (tuned mass) attached to it; making a system as 2-DOF

Basic Equations:

$$w^2 = \frac{k}{m} \quad , \quad c = 2\xi w m$$

$$w_d^2 = \frac{k_d}{m_d} \quad , \quad c_d = 2\xi_d w_d m_d$$

Governing equation of motion is for mass m (primary-structure mass):

$$(1+\overline{m})\ddot{u}+2\xi\omega\dot{u}+\omega^2 u = \frac{p}{m}-\overline{m}\,\ddot{u}_d$$

Governing equation of motion for mass m_d (tuned mass):

$$\ddot{u}_d+2\xi_d\omega_d\dot{u}_d+\omega_d^2 u_d = -\ddot{u}$$

The governing parameter in designing of TMD are m_d, k_d, c_d.

Here we assume the near optimal approximation: $w_d = w$ (i.e. tuning mass damper to fundamental frequency of the structure).

$$k_d = \frac{m_d \times k}{m}$$

Now consider periodic excitation as:

$$p = p^{\wedge} \sin \Omega t$$

Thus the response is given by

$$u = \hat{u}\sin(\Omega t+\delta_1)$$

$$u_d = \hat{u}_d \sin(\Omega t+\delta_1+\delta_2)$$

... $\hat{u}$ = displacement response amplitude, δ = phase shift.

When the loading scenario is the resonant condition i.e. $\Omega = w$.

The solution takes the form:

$$\hat{u} = \frac{\hat{p}}{k\overline{m}}\sqrt{\frac{1}{1+\left(\frac{2\xi}{\overline{m}}+\frac{1}{2\xi_d}\right)^2}}$$ Displacement response amplitude for the primary mass.

$$\hat{u}_d = \frac{1}{2\xi_d}\hat{u}$$ Displacement response amplitude for the secondary mass.

$$\tan\delta_1 = -\left[\frac{2\xi}{\overline{m}}+\frac{1}{2\xi_d}\right]$$

$$\tan\delta_2 = -\frac{\pi}{2}$$

The response of tuned mass damper is out of phase with the response of primary mass. This difference in phase generates damper inertia force causing energy dissipation.

The solution without tuned mass damper is:

$$\hat{u} = \frac{\hat{p}}{k}\left(\frac{1}{2\xi}\right)$$

Thus comparing the solution with and without TMD gives solution in terms of equivalent damping.

$$\hat{u} = \frac{\hat{p}}{k}\left(\frac{1}{2\xi_e}\right)$$

$$\xi_e = \frac{\overline{m}}{2}\sqrt{1 + \left(\frac{2\xi}{\overline{m}} + \frac{1}{2\xi_d}\right)^2}$$

The above equation gives relative contribution of the damper parameters to the total damping and suggests following constraints related with the practical implementation of TMD.

- Increasing the mass ratio $\overline{m}$ i.e. m_d (tuned mass) increases the system damping. However there is a practical limit in increasing the secondary mass or m_d.

Thus selecting the final design requires a compromise for mass constraints.

Following example gives the relation between these parameters.

Example: If $\xi = 0$ and assuming system damping ξ_e= 10 % = 0.1, we obtain the mass ratio $\overline{m}$ = 0.02 i.e. secondary mass addition of 2 % which is typical. But displacement in secondary mass increases 10 times that of primary mass which becomes critical in practical design and desires the provision of sliding of mass.

Chapter 3

Mathematical Modeling and Analytical Parameters

3.1 Modelling of bridge

3.1.1 General

In the present study three steel truss bridges with clear span 45.7 m, 61 m and 76.2 m which are designed for 25 T loading are taken. The data corresponding to truss bridge configurations and member section details has been collected from Research Design and Standards Organization (RDSO) (Ministry of Railways), Lucknow, India. *[12]*.

Modeling and analysis of all models is performed using SAP Computers and Structures version 14.2.2. All bridges are considered to be simply supported at their ends (with one end hinged and other end roller). All other joints in the model are considered as pinned. 2D planar analysis is applied with consideration of plane containing longitudinal axis and vertical axis of bridge. Direct integration linear time history analysis is used for dynamic analysis. Damping was considered uniform for all modes. Basic response parameter considered is the vertical displacement at the centre of the bridge. Equal vertical earthquake ground motion time-history is applied at the end supports.It is chosen specifically for simply supported beam because first mode contributes maximum under vibration and maximum ordinate for first mode is found at the centre of bridge span for simply supported condition.

3.1.2 Material properties of bridge models

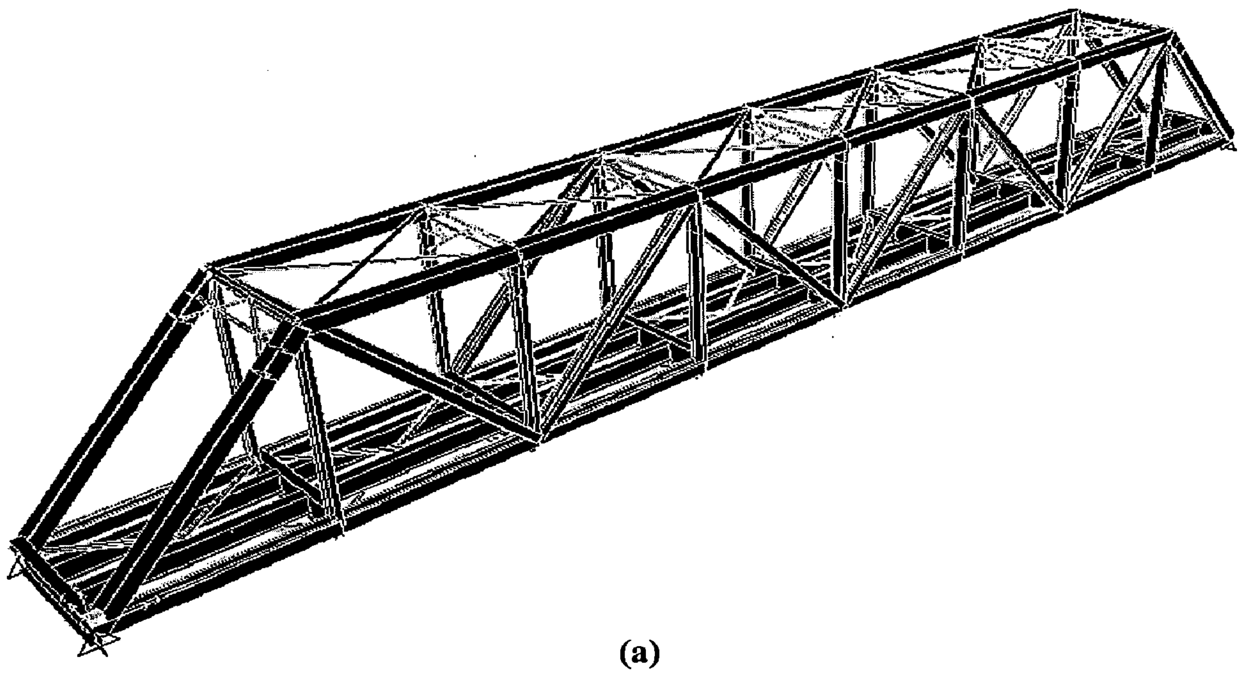
Following are the basic material properties that are considered for all bridge models.

- Material type: Steel ASTM A36
- Density of material: 7850 kg / m3
- Modulus of elasticity = 2.0 E+11 N/m2
- Poisson's Ratio = 0.3
- Material Damping = 2 %

3.1.3 Geometrical properties of bridge models

3.1.3.1 45.7 m span truss bridge

The clear span and effective span of the truss bridge are 45.7 m and 47.24 m respectively. The cross sectional details of the bridge for 25 T category of loading is shown in fig. 3.1 accompanied by Table 3.1.



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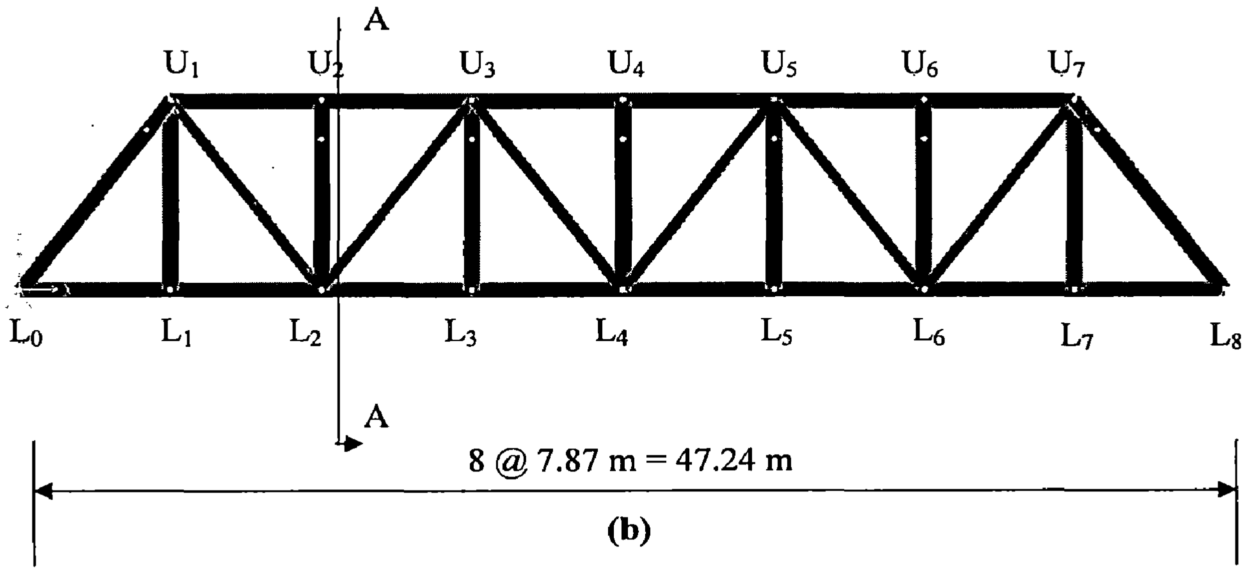

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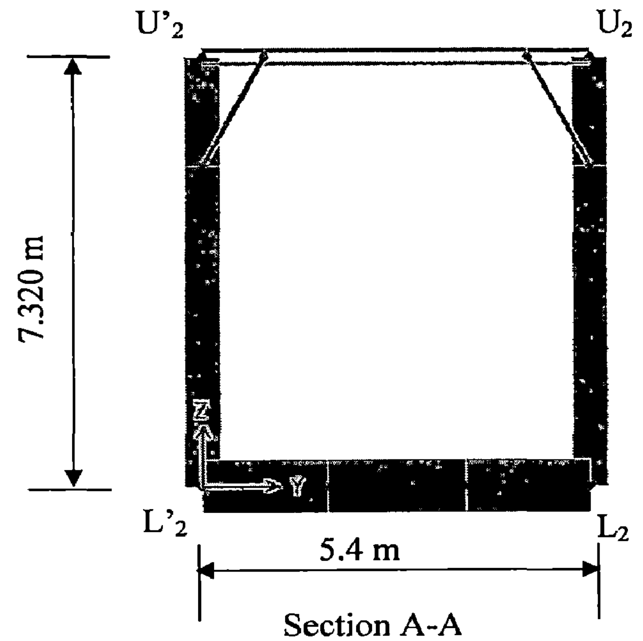

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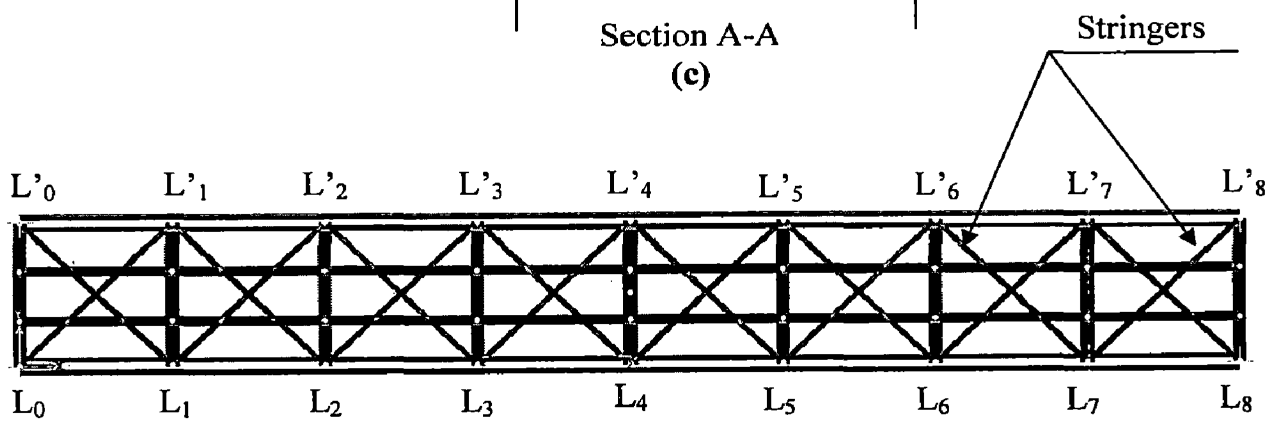

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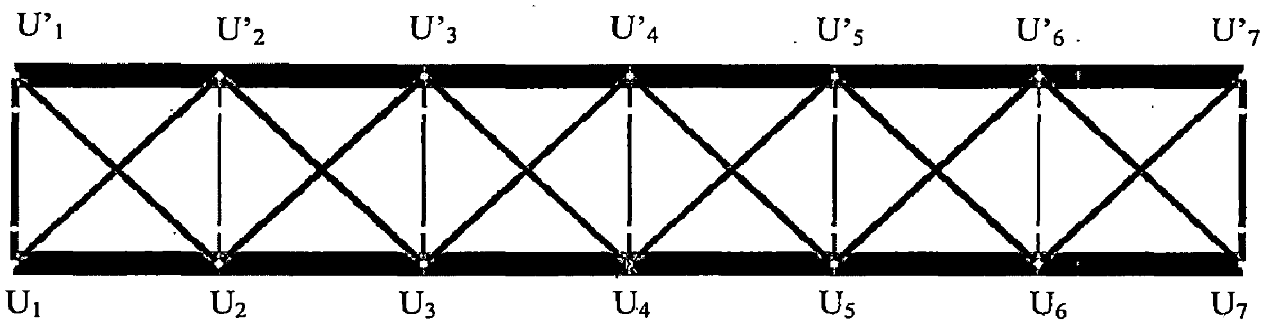


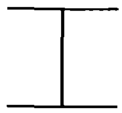
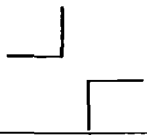
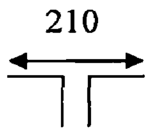
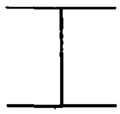
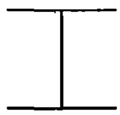
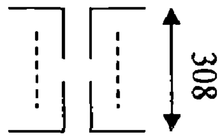
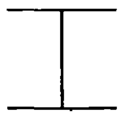
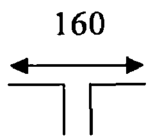
(e)

Figure 3.1 Mathematical model of 45.7 m span truss Bridge (a) Isometric View (b) Front View (c) Cross Sectional View (d) Bottom Chord View (e) Top Chord View

Table 3.1 Member details of 45.7 m span truss bridge for 25T loading route

Sl.No	Member	Description	Section
1	$L_0 - L_1$, $L_1 - L_2$, $L_4 - L_5$, $L_5 - L_6$	Bottom Chord-1	2 [s ISMC 300x90 1 TOP FL.Pl. 460x8 610
2	$L_2 - L_3$, $L_3 - L_4$	Bottom Chord-2	2 [s ISMC 300x90 1 TOP FL.Pl. 460x12 2 ADDL Pls. 240x8 610
3	$U_1 - U_2$, $U_2 - U_3$, $U_3 - U_4$, $U_4 - U_5$	Top Chord	2 [s ISMC 300x90 1 TOP FL.Pl. 460x8 610
4	$L_0 - U_1$, $L_6 - U_5$	Diagonal-1	2 [s ISMC 300x90 1 TOP FL.Pl. 460x12 2 ADDL Pls. 240x8 610
5	$L_2 - U_1$, $L_4 - U_5$	Diagonal-2	2 [s ISMC 300x90 610
6	L_2 - U_3, $L_4 - U_3$	Diagonal-3	2 [s ISMC 250x80 610

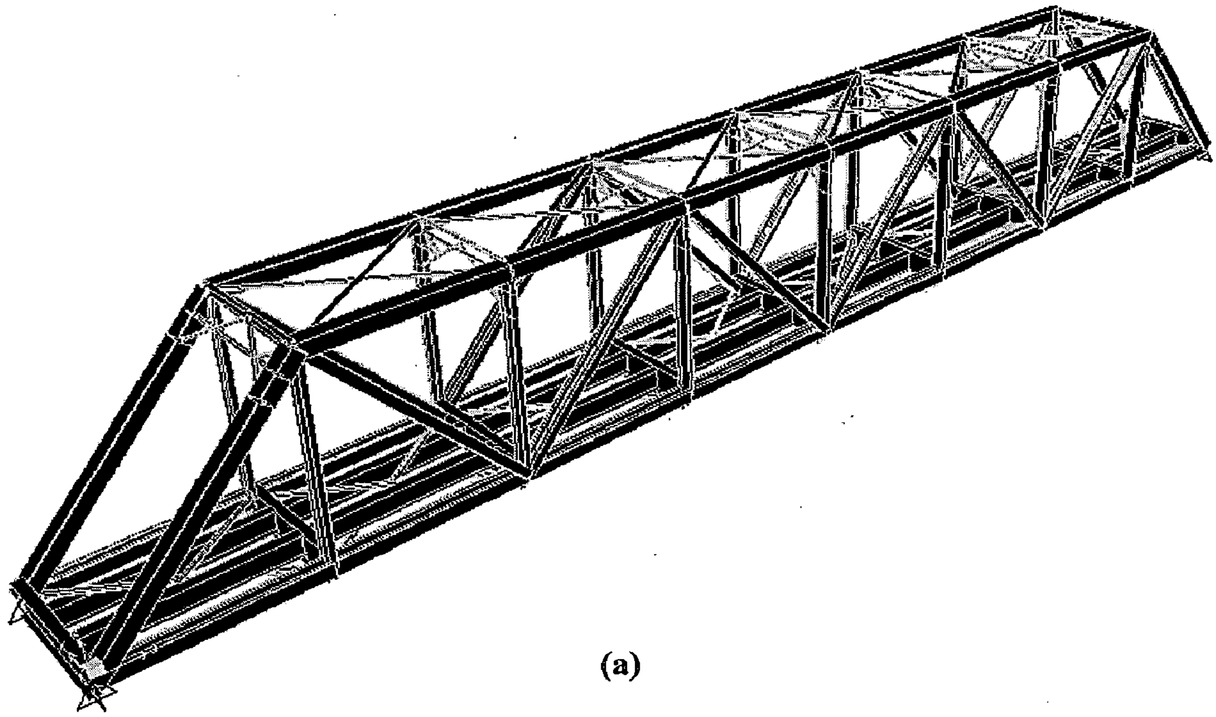
Table 3.1 (Contd.)

7	$U_1 - L_1$, $U_2 - L_2$, $U_3 - L_3$, $U_4 - L_4$, $U_5 - L_5$	Verticals	Web Pl. 610x10 TOP & BOTTOM FLANGE.PLATES. 220x10	
8	Panels:- $L_0 - L'_1$, $L_1 - L'_2$, $L_2 - L'_3$, $L_4 - L'_5$, $L_5 - L'_6$	Bottom Lateral bracing	2 Ls 100x100x8	
9	Panels:- $U_1 - U_2$, $U_2 - U_3$, $U_3 - U_4$, $U_4 - U_5$	Top Lateral Bracing	2 Ls 100x100x8	
10	$L_0 - L'_0$, $L_1 - L'_1$, $L_2 - L'_2$, $L_3 - L'_3$, $L_4 - L'_4$, $L_5 - L'_5$, $L_6 - L'_6$	Cross Girders	WEB Pl. 864x10 TOP & BOTT.FL.Pls. 410x18	
11	$U_1 - U'_1$, $U_5 - U'_5$	Portal Girders	WEB Pl. 350x8 TOP & BOTT.FL.Pls. 230x10	
12	$U_2 - U'_2$, $U_3 - U'_3$, $U_4 - U'_4$	Sway Girders	4 Ls 75x75x10 Lacing Flat 65x10	
13		Stringers	2 Fl.Pl. 320x16 1 Web Pl. 718x10	
14	All Sway and Portal Bracing		2Ls 75x75x10	

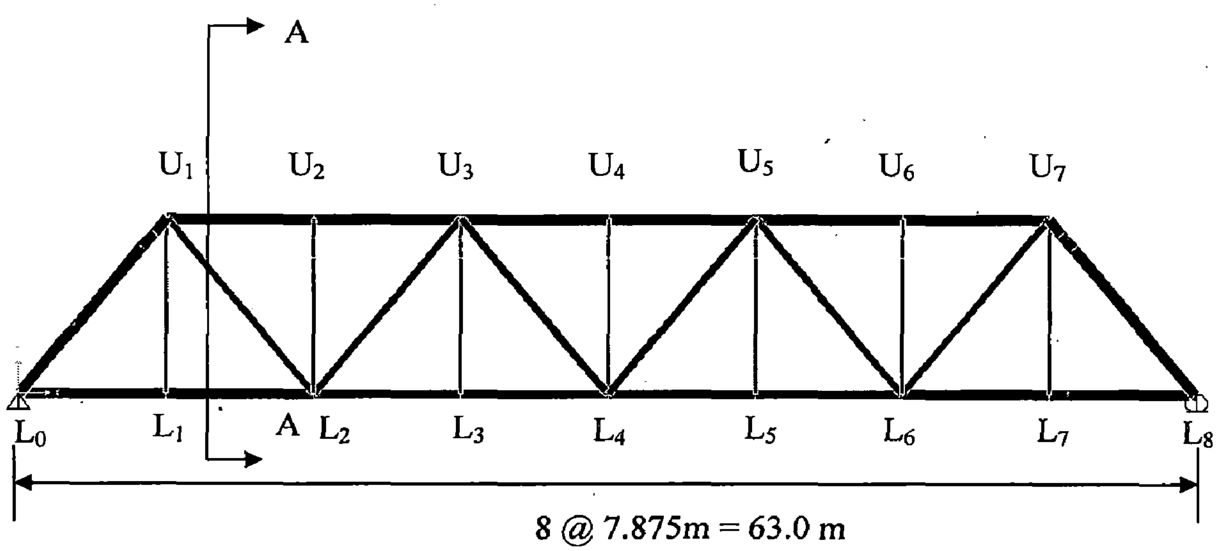
Note: All dimensions are in millimeter

3.1.3.2 61.0 m span truss bridge

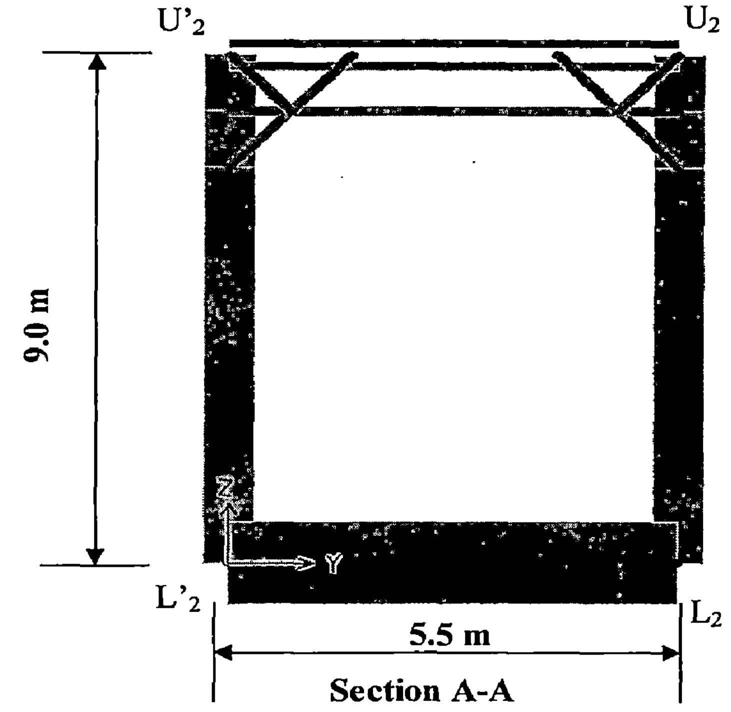
The clear span and effective span of the truss bridge are 61.0 m and 63.0 m respectively. The cross sectional detail of the bridge for 25 T category of loading is shown in fig. 3.2 accompanied by Table 3.2.



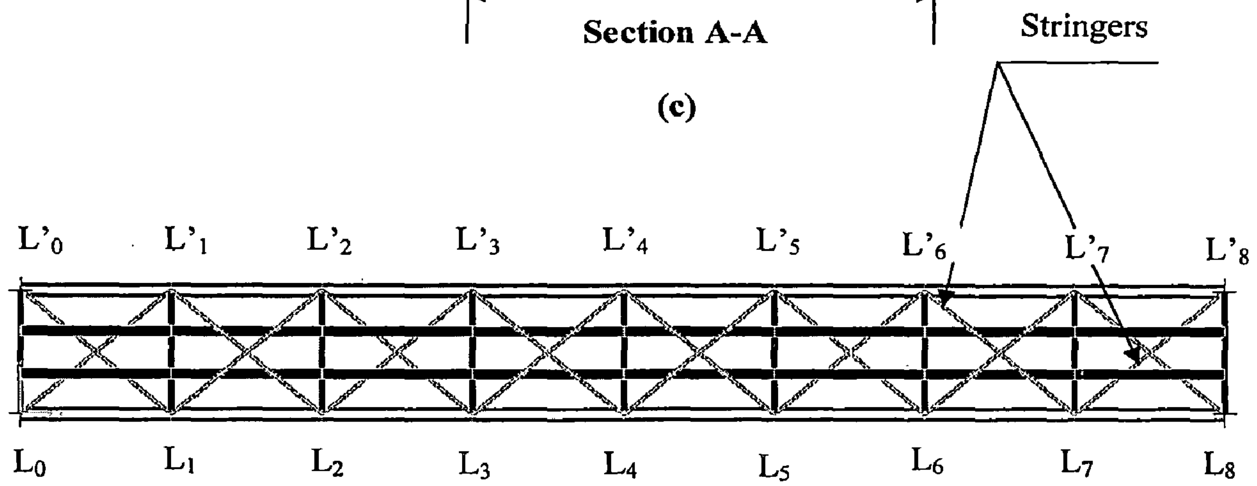
(a)



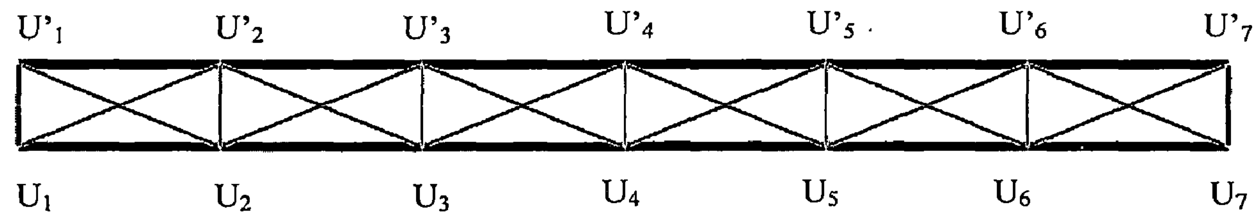
(b)



(c)



(d)



(e)

Figure 3.2 Mathematical model of 61.0 m span truss bridge (a) Isometric View (b) Front View (c) Cross Sectional View (d) Bottom Chord View (e) Top Chord View

Table 3.2 Member details of 61.0 m span truss bridge for 25T loading route

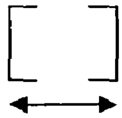
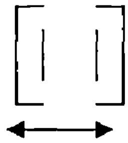
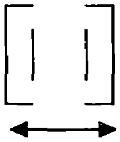
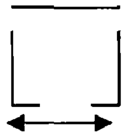
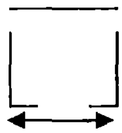
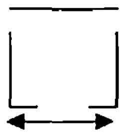
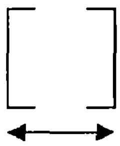
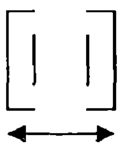
Sl.No	Member	Description	Section
1	$L_0 - L_1$, $L_1 - L_2$,	Bottom Chord-1	2 SIDE Pls.620x20 4 Pls. 150x20  610
2	$L_2 - L_3$, $L_3 - L_4$, $L_6 - L_7$, $L_7 - L_8$	Bottom Chord-2	2 SIDE Pls.625x25 4 Pls. 150x25 2 ADDL Pls. 500x25  610
3	$L_4 - L_5$, $L_5 - L_6$	Bottom Chord-3	2 SIDE Pls.620x30 4 Pls. 150x30 2 ADDL Pls. 500x25  610
4	$U_1 - U_2$, $U_2 - U_3$,	Top Chord-1	2 SIDE Pls.620x12 1 TOP FL.Pl. 634x16 2 Fl Pls. 150x12  610
5	$U_3 - U_4$, $U_4 - U_5$, $U_5 - U_6$, $U_6 - U_7$	Top Chord-2	2 SIDE Pls.620x20 1 TOP FL.Pl. 634x16 2 Fl Pls. 150x16  610
6	$L_0 - U_1$,	End Diagonal	2 SIDE Pls.620x20 1 TOP FL.Pl. 634x16 2 Fl Pls. 150x20  610
7	L_2 - U_1,	Diagonal-1	2 SIDE Pls.400x20 4 Pls. 150x20  610
8	$L_2 - U_3$, $L_8 - U_7$	Diagonal-2	2 [s ISMC 400x100 2 ADDL Pls. 300x10  610

Table 3.2 (Contd.)

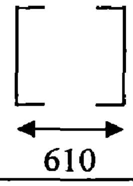
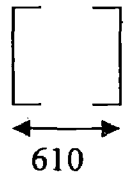
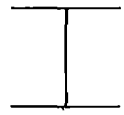
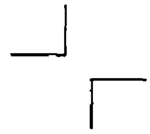
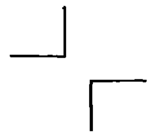
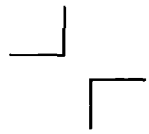
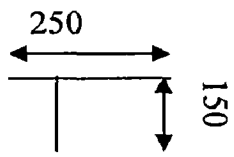
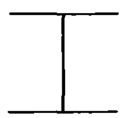
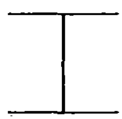
9	$L_4 - U_3$, $L_6 - U_7$	Diagonal-3	2 SIDE Pls.400x16 4 Pls. 150x10 
10	$L_4 - U_5$, $L_6 - U_5$	Diagonal-4	2 SIDE Pls.400x10 4 Pls. 150x10 
11	$U_1 - L_1$, $U_2 - L_2$, $U_3 - L_3$, $U_4 - L_4$, $U_5 - L_5$, $U_6 - L_6$, $U_7 - L_7$,	Verticals	WEB Pl. 590x10 TOP & BOTT.FL.Pls. 280x10 
12	Panels:- $L_0 - L_1$, $L_1 - L_2$,	Bottom Lateral Bracing-1	2 Ls 130x130x12 
13	Panels:- $L_2 - L_3$, $L_3 - L_4$, $L_6 - L_7$, $L_7 - L_8$	Bottom Lateral Bracing-2	2 Ls 100x100x10 
14	Panels:- $L_4 - L_5$, $L_5 - L_6$	Bottom Lateral Bracing-3	2 Ls 75x75x10 
15	Panels:- $U_1 - U_2$, $U_2 - U_3$, $U_3 - U_4$, $U_4 - U_5$, $U_5 - U_6$, $U_6 - U_7$,	Top Lateral Bracing	ISHT 150 
16	$L_0 - L_0'$, $L_1 - L_1'$, $L_2 - L_2'$, $L_3 - L_3'$, $L_4 - L_4'$, $L_5 - L_5'$, $L_6 - L_6'$, $L_7 - L_7'$, $L_8 - L_8'$	Cross Girders	WEB Pl. 1400x12 TOP & BOTT.FL.Pls. 400x20 
17	$U_1 - U'_1$,	Portal Girders	WEB Pl. 430x10 TOP & BOTT.FL.Pls. 200x10 

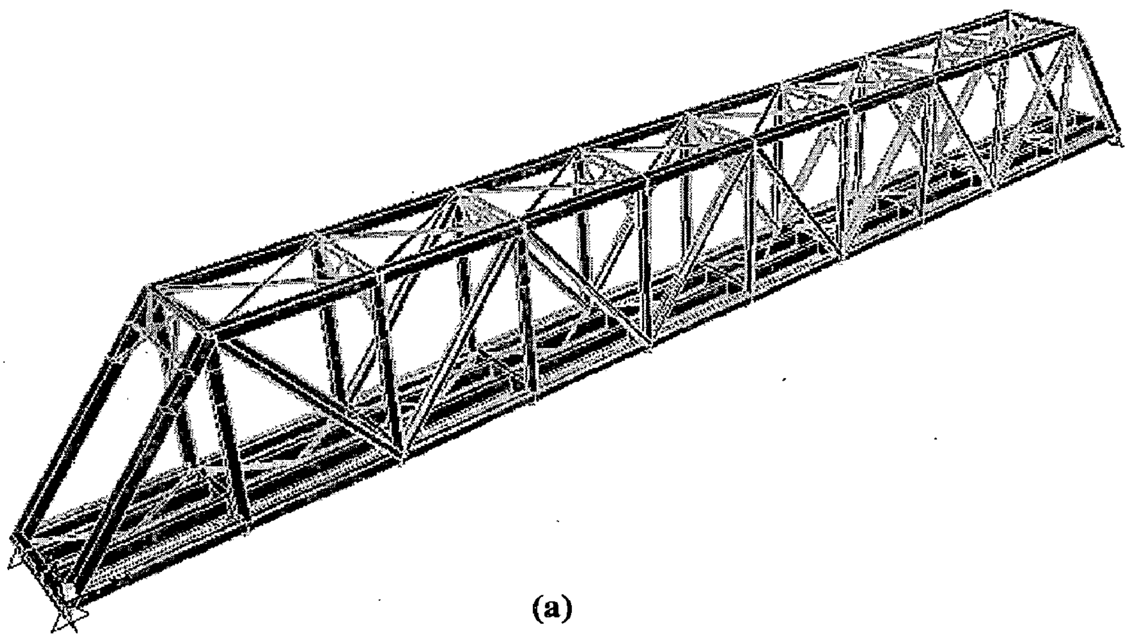
Table 3.2 (Contd.)

18	$U_2 - U'_2$, U_3 -U'_3, $U_4 - U'_4$, $U_5 - U_5'$, $U_6 - U_6'$, $U_7 - U_7'$,	Sway Girders	2 ISNT 150 (636)
19		Stringers	WEB Pl. 900x12 TOP & BOTT.FL.Pls. 450x20
20	All Portal Bracing		2Ls 100x100x10
21	All Sway Bracings		2Ls 75x75x8

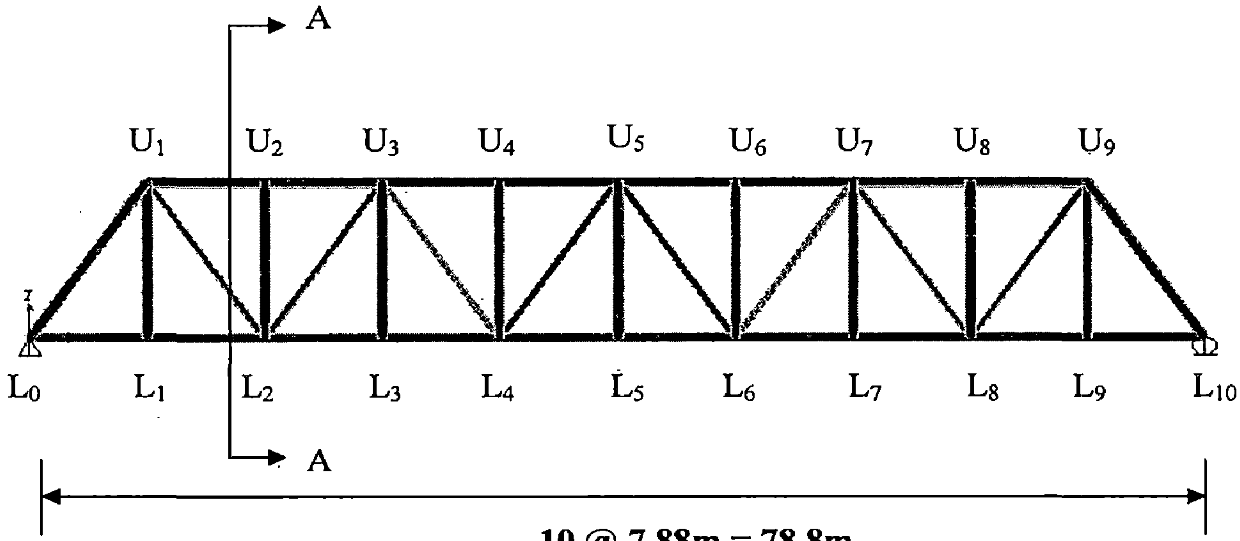
Note: All dimensions are in millimeter

3.1.3.3 76.2 m span welded truss bridge

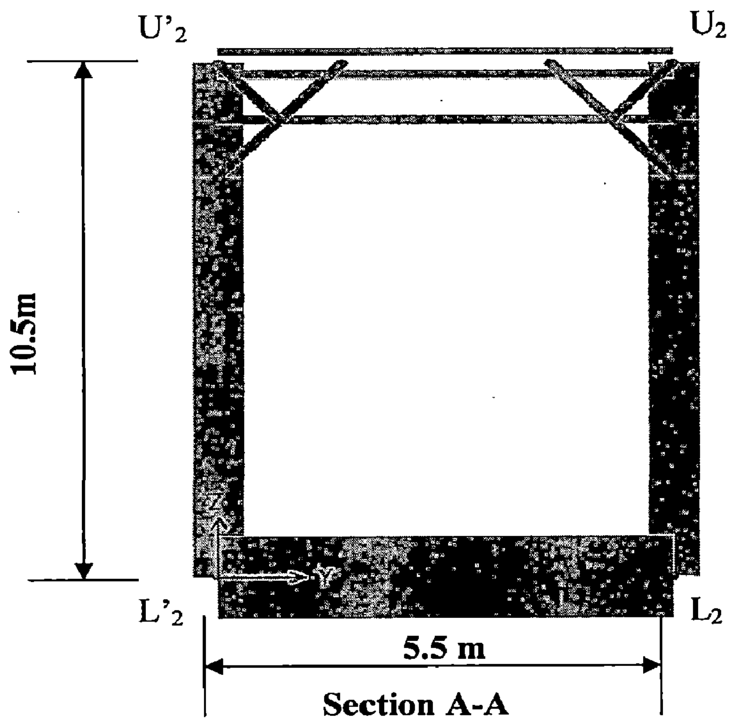
The clear span and effective span of the truss bridge are 76.2 m and 78.8 m respectively. The cross sectional details of the bridge for 25 T category of loading are shown in fig. 3.3 accompanied by Table 3.3.



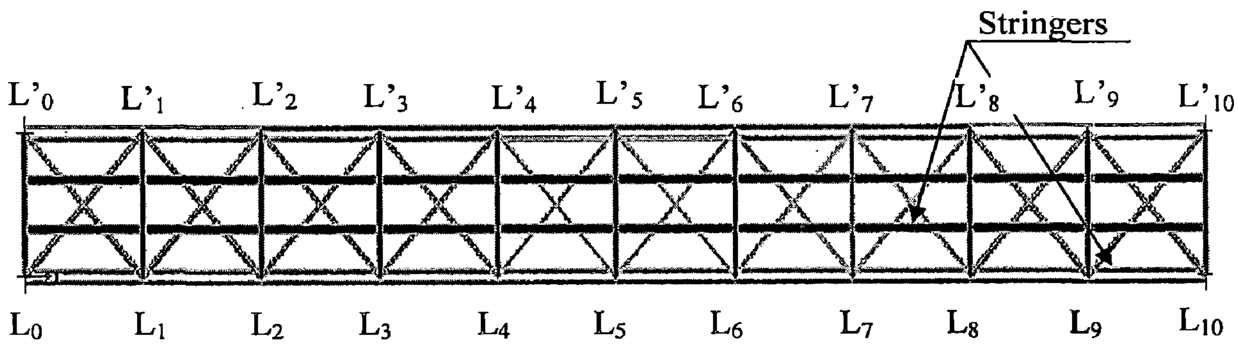
(a)


A
U1
U2
U3
U4
U5
U6
U7
U8
U9
L0
L1
L2
L3
L4
L5
L6
L7
L8
L9
L10
A
10 @ 7.88m = 78.8m

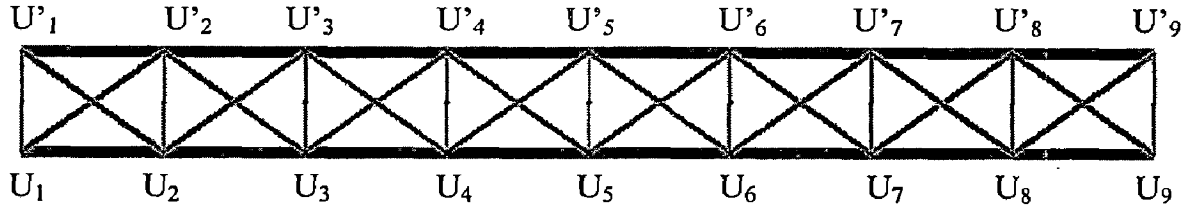
(b)


U'2
U2
10.5m
L'2
L2
5.5 m
Section A-A

(c)


Stringers
L'0
L'1
L'2
L'3
L'4
L'5
L'6
L'7
L'8
L'9
L'10
L0
L1
L2
L3
L4
L5
L6
L7
L8
L9
L10

(d)



(e)

Figure 3.3 Mathematical model of 76.2m span truss bridge (a) Isometric View (b) Front View (c) Cross Sectional View (d) Bottom Chord View (e) Top Chord View

Table 3.3 Member details of 76.2 m span truss bridge for 25T loading route

Sr.No	Member	Description	Section
1	$L_0 - L_1$, $L_1 - L_2$, $L_8 - L_9$, $L_9 - L_{10}$	Bottom Chord-1	2 SIDE Pls.620x20 4 Pls. 150x20 610
2	$L_2 - L_3$, $L_3 - L_4$, $L_6 - L_7$, $L_7 - L_8$	Bottom Chord-2	2 SIDE Pls.625x25 4 Pls. 150x25 2 ADDL Pls. 500x25 610
3	$L_4 - L_5$, $L_5 - L_6$	Bottom Chord-3	2 SIDE Pls.620x30 4 Pls. 150x30 2 ADDL Pls. 500x25 610
4	$U_1 - U_2$, $U_2 - U_3$, $U_7 - U_8$, $U_8 - U_9$	Top Chord-1	2 SIDE Pls.620x12 1 TOP FL.Pl. 634x16 2 Fl Pls. 150x12 610
5	$U_3 - U_4$, $U_4 - U_5$, $U_5 - U_6$, $U_6 - U_7$	Top Chord-2	2 SIDE Pls.620x20 1 TOP FL.Pl. 634x16 2 Fl Pls. 150x16 610
6	$L_0 - U_1$, $L_{10} - U_9$	End Diagonal	2 SIDE Pls.620x20 1 TOP FL.Pl. 634x16 2 Fl Pls. 150x20 610

Table 3.3 (Contd.)

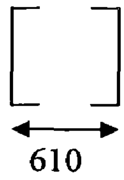
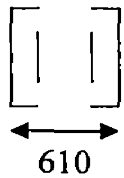
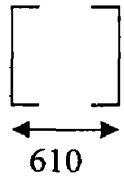
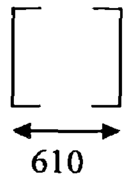
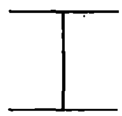
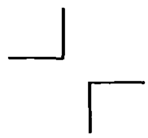
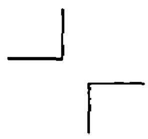
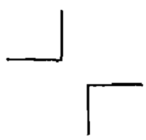
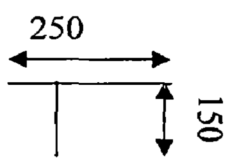
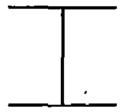
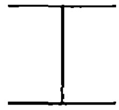
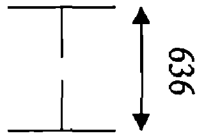
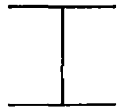
7	$L_2 - U_1$, $L_8 - U_9$	Diagonal-1	2 SIDE Pls.400x20 4 Pls. 150x20	
8	$L_2 - U_3$, $L_8 - U_7$	Diagonal-2	2 [s ISMC 400x100 2 ADDL Pls. 300x10	
9	$L_4 - U_3$, $L_6 - U_7$	Diagonal-3	2 SIDE Pls.400x16 4 Pls. 150x10	
10	$L_4 - U_5$, $L_6 - U_5$	Diagonal-4	2 SIDE Pls.400x10 4 Pls. 150x10	
11	$U_1 - L_1$, $U_2 - L_2$, $U_3 - L_3$, $U_4 - L_4$, $U_5 - L_5$, $U_6 - L_6$, $U_7 - L_7$, $U_8 - L_8$, $U_9 - L_9$,	Verticals	WEB Pl. 590x10 TOP & BOTT.FL.Pls. 280x10	
12	Panels:- $L_0 - L_1$, $L_1 - L_2$, $L_8 - L_9$, $L_9 - L_{10}$	Bottom Lateral Bracing-1	2 Ls 130x130x12	
13	Panels:- $L_2 - L_3$, $L_3 - L_4$, $L_6 - L_7$, $L_7 - L_8$	Bottom Lateral Bracing-2	2 Ls 100x100x10	
14	Panels:- $L_4 - L_5$, $L_5 - L_6$	Bottom Lateral Bracing-3	2 Ls 75x75x10	
15	Panels:- $U_1 - U_2$, $U_2 - U_3$, $U_3 - U_4$, $U_4 - U_5$, $U_5 - U_6$, $U_6 - U_7$, $U_7 - U_8$, $U_8 - U_9$	Top Lateral Bracing	ISHT 150	

Table 3.3 (Contd.)

16	$L_0 - L_0'$, $L_1 - L_1'$, $L_2 - L_2'$, $L_3 - L_3'$, $L_4 - L_4'$, $L_5 - L_5'$, $L_6 - L_6'$, $L_7 - L_7'$, $L_8 - L_8'$ $L_9 - L_9'$, $L_{10} - L_{10}'$	Cross Girders	WEB Pl. 1400x12 TOP & BOTT.FL.Pls. 400x20 
17	$U_1 - U'_1$, $U_9 - U'_9$	Portal Girders	WEB Pl. 430x10 TOP & BOTT.FL.Pls. 200x10 
18	$U_2 - U'_2$, $U_3 - U'_3$, $U_4 - U'_4$, $U_5 - U_5'$, $U_6 - U_6'$, $U_7 - U_7'$, $U_8 - U_8'$	Sway Girders	2 ISNT 150 
19		Stringers	WEB Pl. 900x12 TOP & BOTT.FL.Pls. 450x20 
20	All Portal Bracing		2Ls 100x100x10 
21	All Sway Bracings		2Ls 75x75x8 

Note: All dimensions are in millimeter

3.2 Modeling of TMD system

Tuned Mass Damper system, (a secondary system) consists of a spring, damper and mass, which is attached to bridge (a primary structural system). Parameters for modeling of spring, damper and mass depends on the bridge properties and properties of excitation force (here earthquake time history) that are used for analysis.

Attached Mass: Mass of tuned mass damper system is considered to be about 5 % of the bridge *[1]*. For multiple tuned mass damper system mass of each tuned mass is kept constant with their total mass approximately equal to 5 % of bridge. Mass is attached to the free (hanging) end of the spring as shown in fig. 3.4.

Damping of damper: Damping of tuned mass damper considered is 25 % of critical damping. Damping helps much in reducing the response in bridge and substantially reduces the displacement of the tuned mass. But higher damping can render tuned mass damper ineffective in reducing the response of primary system and practically non-effective. Damper is attached to primary system at one end and other end is attached to mass as shown in fig. 3.4.

Stiffness of spring: Stiffness of spring largely depends on the predominating frequencies of different time-histories that it has to get tune with to make maximum reduction in response of bridge. Since it is not possible to tune TMD system to each and every earthquake time-history, some specific frequencies, based on dominating frequencies of given time-histories, are used for modeling. Dominating frequencies are found out using Fourier transform of each given time history. Circular frequency of single tmd can be found out using following equation:

$$w^2 = k / m$$

w = circular frequency of TMD, k = stiffness of spring of TMD, m = mass of TMD

Position of TMD system: Maximum displacement of simply supported beam is at centre. Thus TMD is to be attached at the point of maximum displacement to reduce the response at that point.

Attachment of TMD: Only vertical axial deformation and rotation in plane of consideration (i.e. about transverse axis of bridge) is allowed. One end of spring (plus damper), is attached (pinned) to centre of beam and other end is hanged upon by tuned mass as shown in fig. 3.4

Number of TMDs to be used in a TMD system: Single tuned mass damper poses only one frequency characteristic and thus is effective (and highly sensitive) for very narrow range of excitation frequency. That means TMD can only be effective for the time history possessing the near-about frequency as that of it. But it is not possible that all earthquake time-histories have the same dominating frequency. In fact they will be varying in broad range. So to cater for broad range of frequency, more number of TMDs are to be used as a single system, with each TMD possessing its characteristic frequency.

In view of these basic concepts, single TMD is used to show that frequency of the TMD should be based on the dominating excitation frequency. Further, in the study, 3 TMD system and 5 TMD system are used for different time histories for all 3 models to show the overall response reduction.

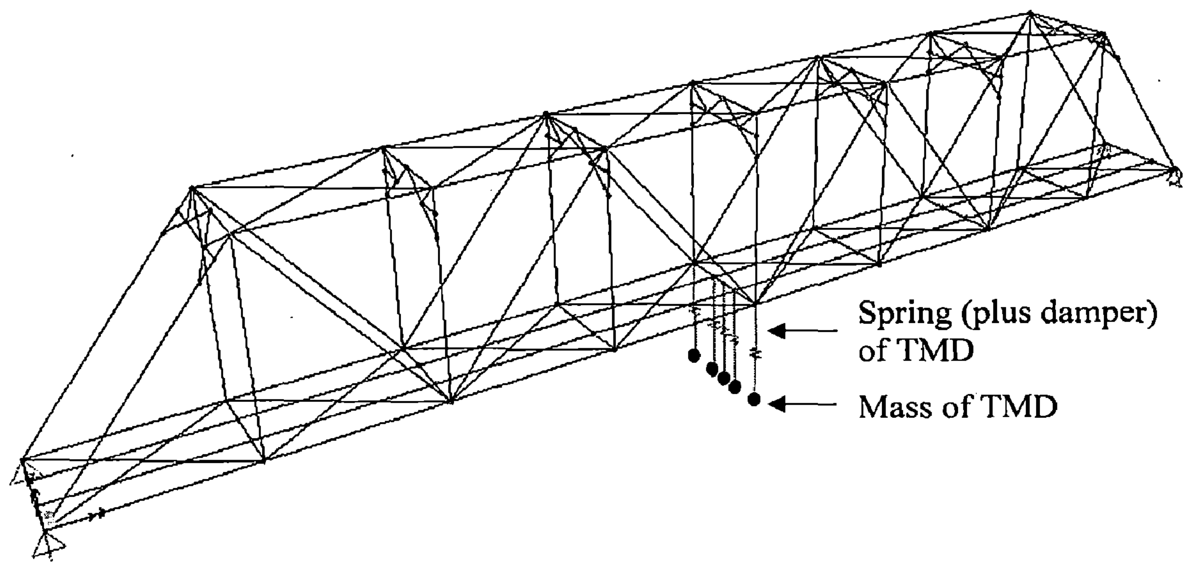


Figure 3.4 Arrangement of TMD system containing 5 TMD with the bridge model.

3.3 Loadings

3.3.1 General

Current study deals with the Uttarakhand region. For this, earthquake time-histories recorded at 15 different stations of this region are taken. Also Fourier spectrum for each time history is generated to get its frequency characteristic which gives the range of dominating excitation frequency. Table 3.4 shows the dominating frequencies of all considered time-histories. Then normalized response spectrum with 5 % damping is calculated for each time-history calculated. Finally an average normalized response spectra is plot by taking arithmetic mean of each response spectrum.

3.3.2 Time-histories (Recorded)

Following are some of the time histories of Uttarakhand region and their respective Fourier spectra giving its frequency characteristics.

1. Garsain (21.09.2009 09:43:54.384) (30.9 N 79.1 E) :

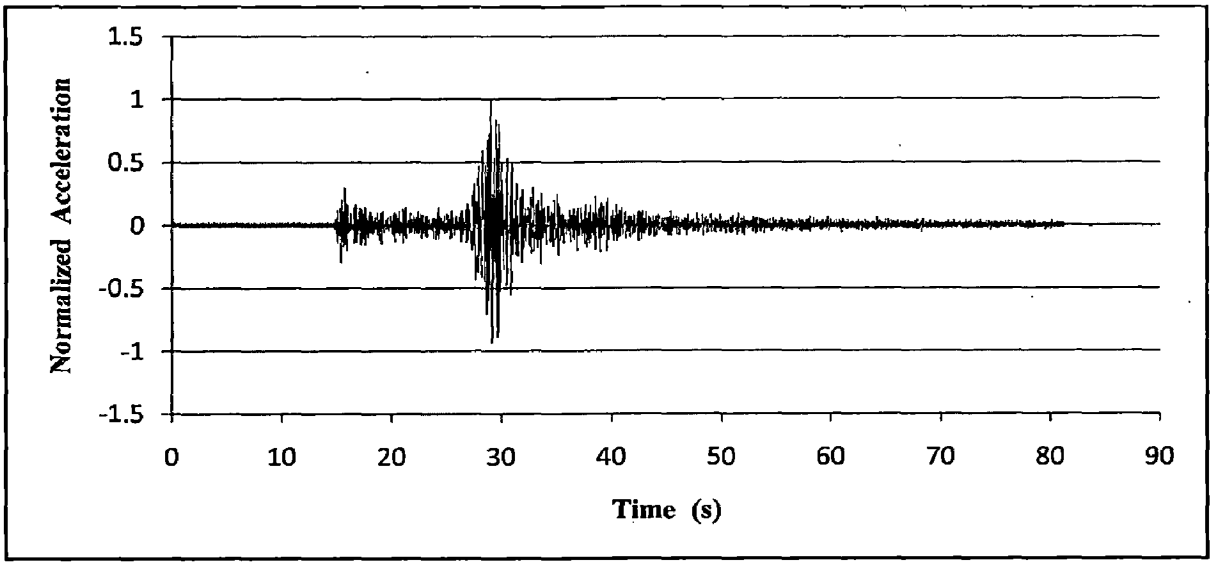


Figure 3.5: Normalized time – history (Garsain station):

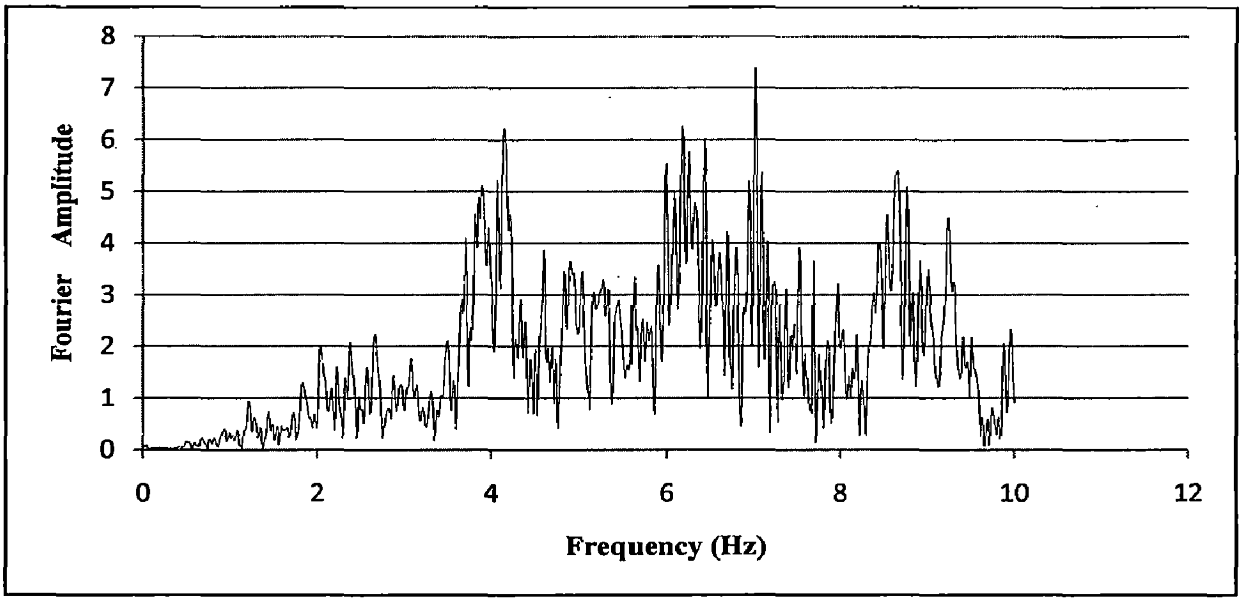


Figure 3.6: Fourier spectra (Garsain station):

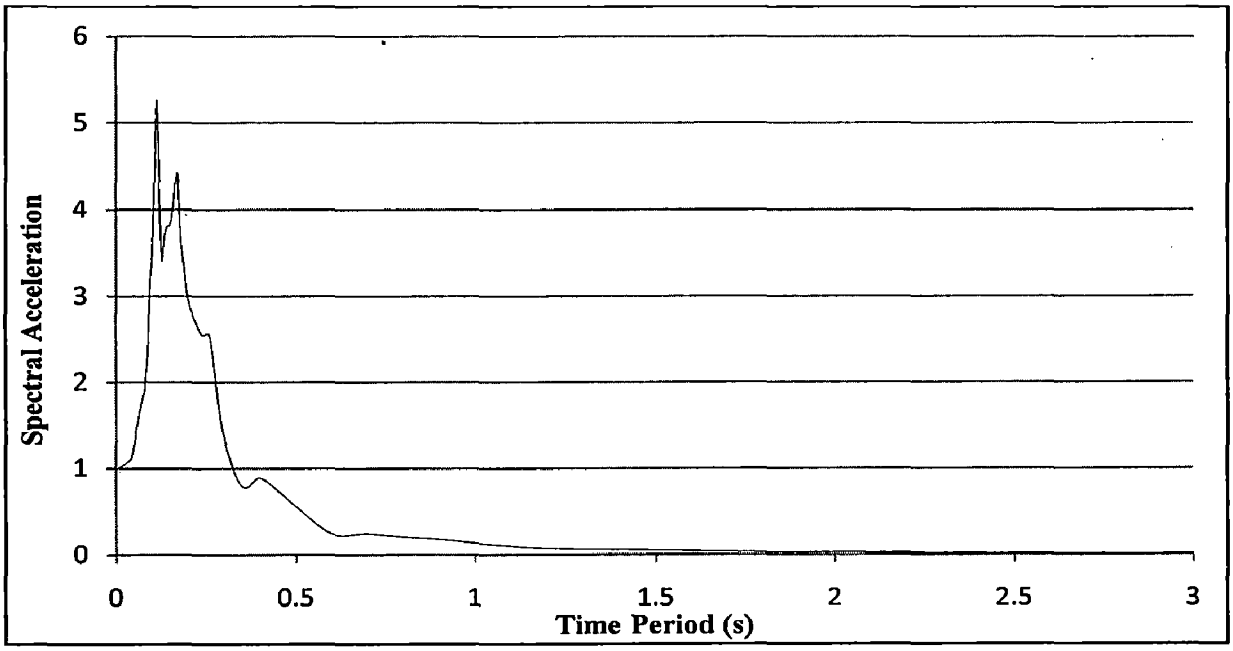


Figure 3.7: Normalized Response Spectra (Garsain station):

2. Ghanshali (21.09.2009 09:43:54.889) (30.434 N 78.656 E):

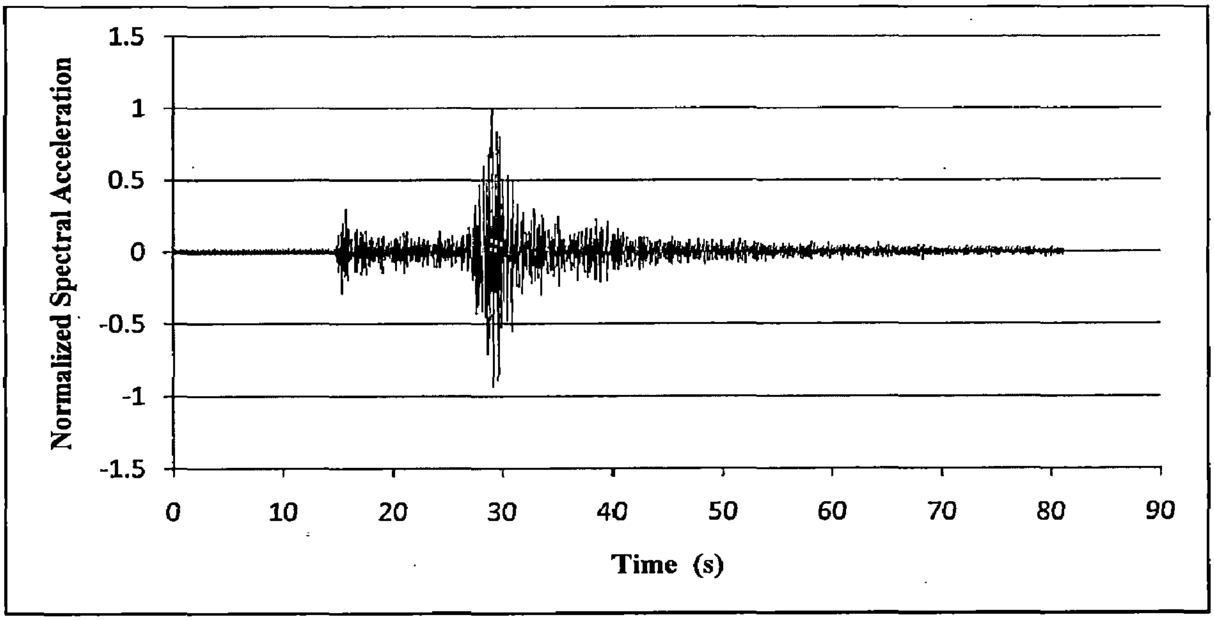


Figure 3.8: Normalized time – history (Ghanshali station):

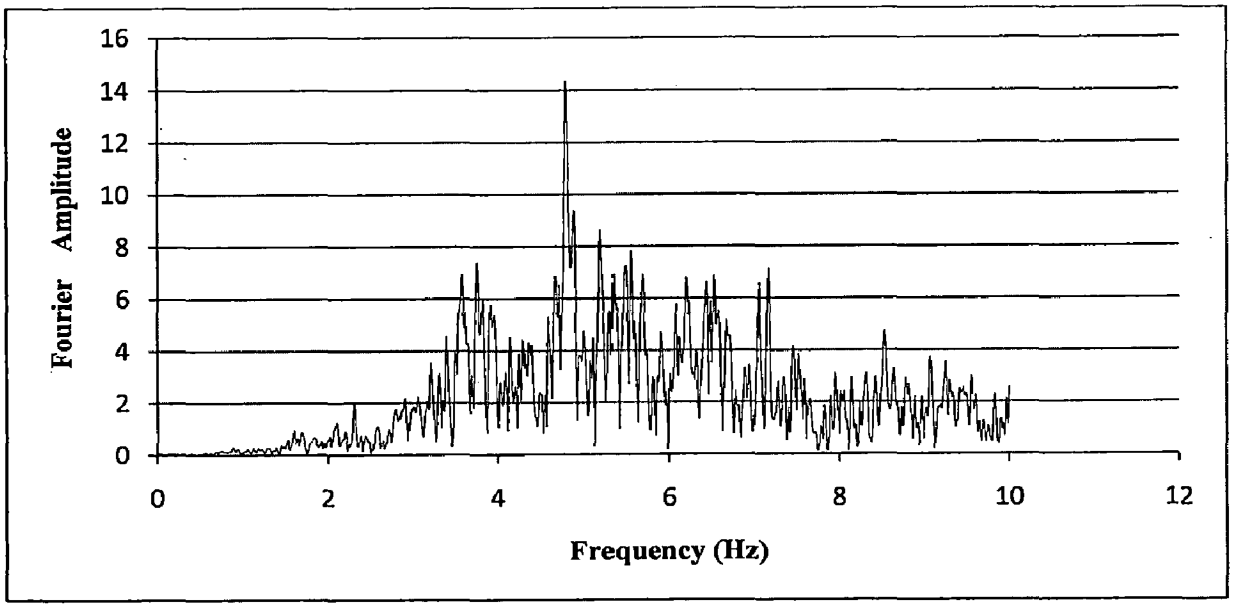


Figure 3.9: Fourier spectra (Ghanshali station):

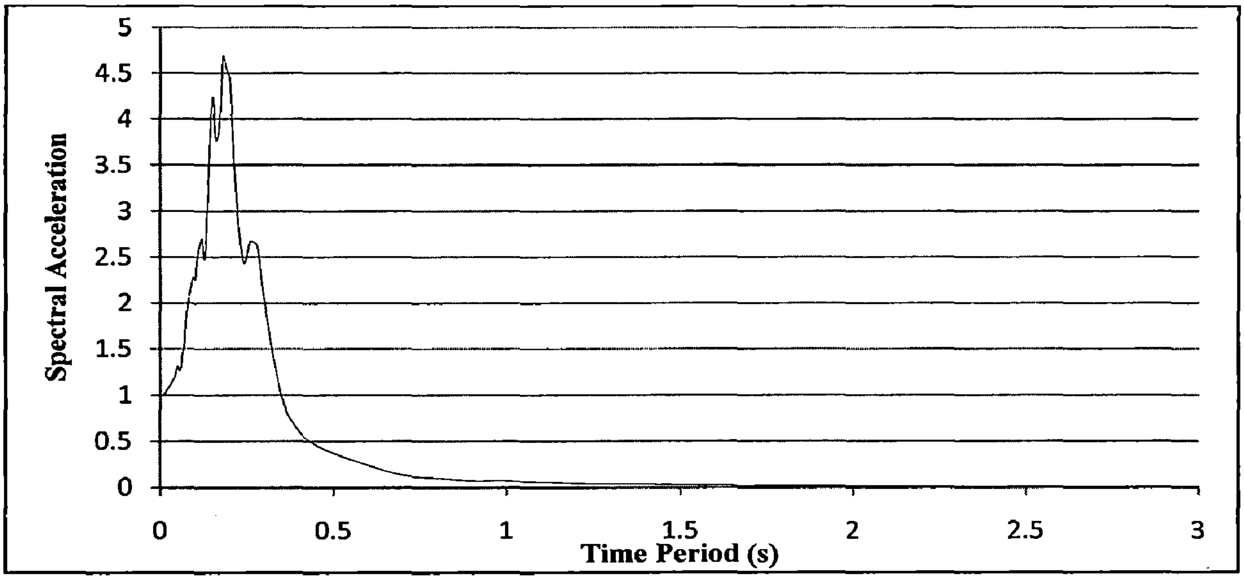


Figure 3.10: Normalized Response Spectra (Ghanshali station):

3.3.3 Average Response Spectra:

Based on 15 different time-histories of Garhwal region, an average normalized response spectra was calculated. It is the average of all the response spectrum developed for each time-history. Following fig. 3.10 shows the average response spectra compared with IS: 1893-2002 code response spectra. We can see that average response spectra is dominating codal response spectra over 0.1 to 0.4 sec range.

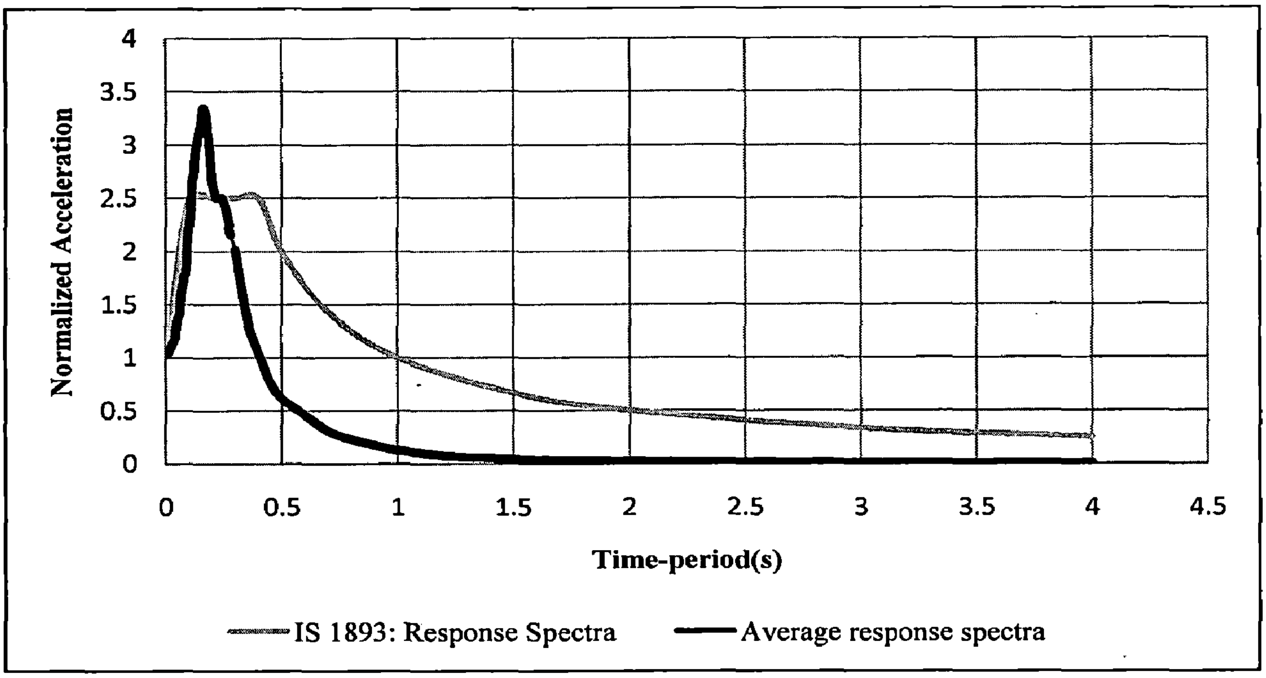


Figure 3.11 Average Response spectra and IS: 1893:2002 Code Response Spectra

3.3.4 Spectrum Compatible time-history:

Usually it is not possible to get an earthquake record at a specific site. And even in presence of record, it is never obvious that we expect the same earthquake in the future. Now earthquake resistant analysis and design may not be reliable based on few numbers of time-histories. Thus some synthetic time-histories need to be generated in such cases *[7]*.

For generating spectrum compatible time-history, SPEC software, developed by Prof. Ashok Kumar, is used *[7]*. Such time-history is generated from the target (already established) response spectra and duration of time-history. The rise and decay time for the desired time-history can be calculated using an in-built envelope function or may be input through the terminal. In present case, Average Normalized Response Spectra that is calculated above is taken as target acceleration spectrum. Duration is assumed to be of 40 seconds. Attack and decay time are taken as 5 and 10 seconds respectively.

Based on these parameters four time-histories namely Th101, Th202, Th303 and Th404 are generated. Again, to know the frequency characteristics of these time-histories are found by plotting their Fourier spectrum for all generated time-histories is found to obtain their frequency characteristic. Shown below are the same

Th101:

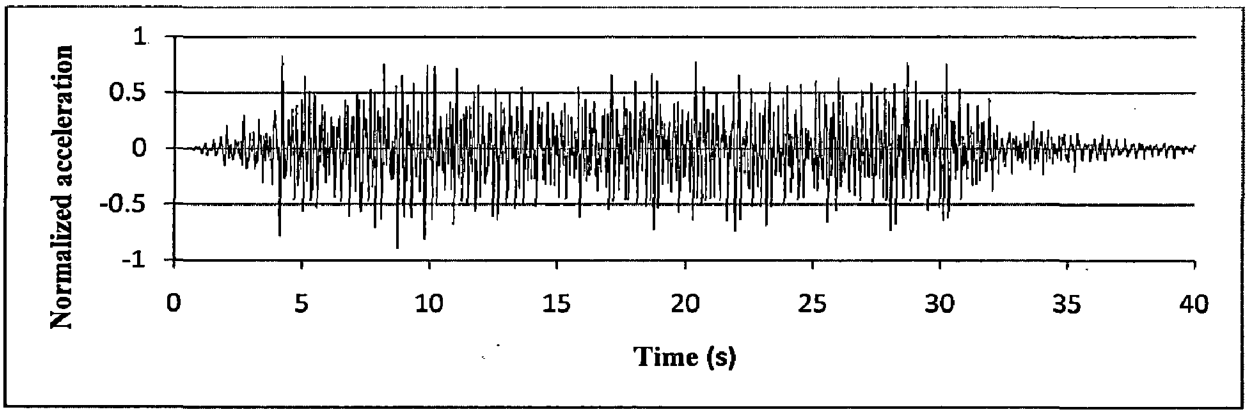


Figure 3.12 Time-history (Th101)

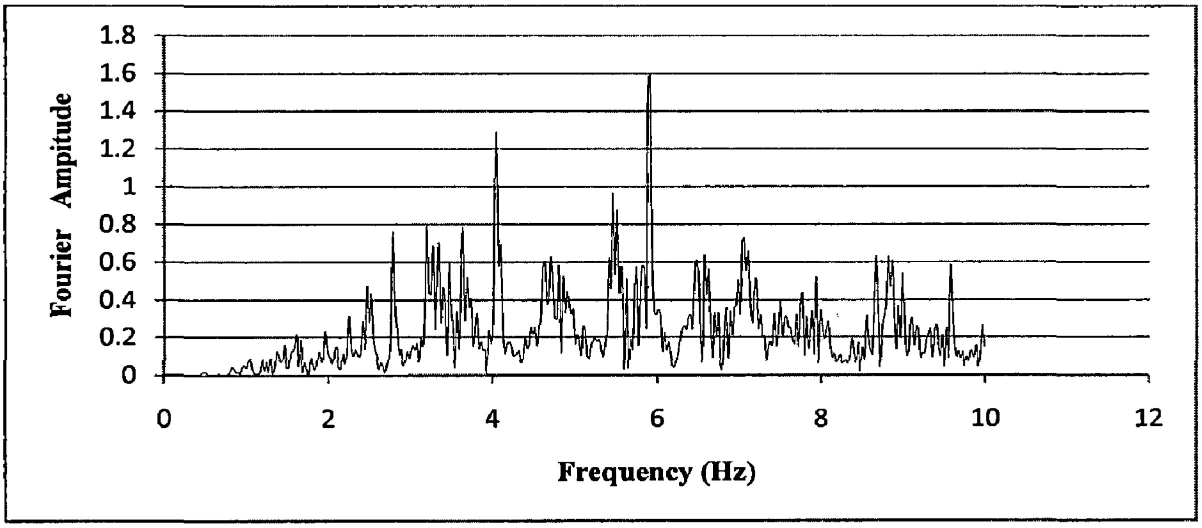


Figure 3.13 Fourier Spectrum (Th101):

Th202:

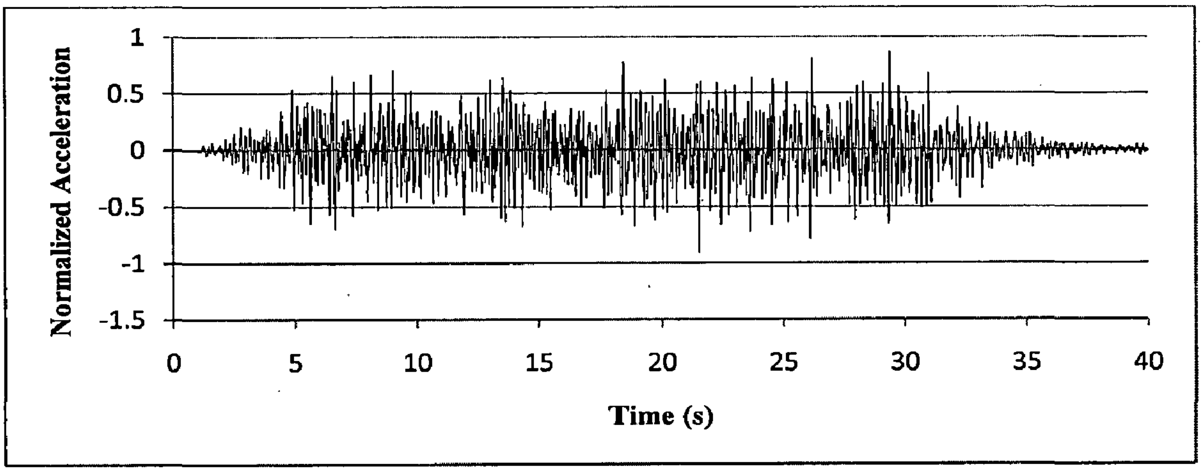


Figure 3.14 Time-history (Th202)

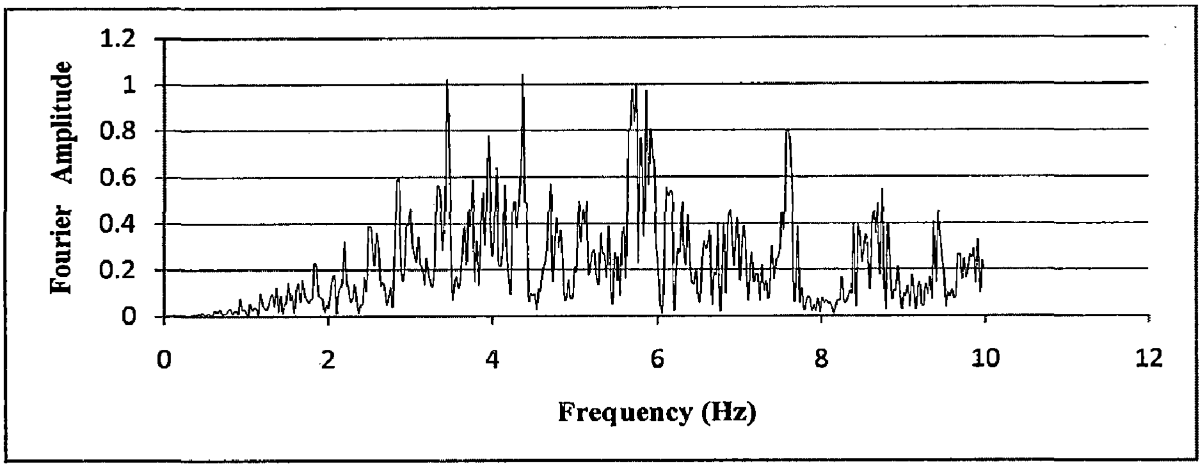


Figure 3.15 Fourier Spectrum (Th202)

3.3.5 Dominating frequencies for different time-histories:

Table 3.4 Dominating frequencies of various time-histories.

Time history	Frequencies Dominating (Hz)												
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	12-15	15-18	18-20
Th101			**4**		**5.8**								
Th202			**3.7**	**4.3**	**5.7**								
Th303			4	**5**	**5.9**								
Th404			3.7		5.2	**6.8**	**7.9**						
Bageshwar			3.7		5.2				**9.1**				
Barkot			**3.6**			6.5							
Chamoli	**1.5**		**3.5**										
Champavat					5.9		**7.8**						
Dhanaulti		3		**4.7**			7.2						
Garsain				**4.1**		6.2	**7**						
Ghanshali				**4.7**									
Jubbal						**6.5**	**7**						
Kapkal			**4**	**4.5**									
Nathpa				**4.1**									
Pauri	2			**4.8**									
Rampur		**3**	**3.6**										
Rudraprayag					5.7	6.8						**16**	**19**
Tehri								**8.2**	**9**				
Uttarkashi		2.5	**3.2**	4.2									

3.4 Analysis of Bridge Model

Linear direct integration time history analysis has been applied for the analysis of all models. SAP (analysis software) uses Hughes – Hilbert – Taylor Method for direct integration. *[13]*

3.4.1 Hughes – Hilbert – Taylor Method *[14]*:

The Hilber-Hughes-Taylor (HHT) method (also known as the alpha-method) is widely used in the structural dynamics community for the numerical integration of a linear set of second order differential equations. It is a type of implicit method.

Provided the finite element approach is linear, the equations of motion assume the form

$$\mathbf{M}\ddot{\mathbf{q}} + \mathbf{C}\dot{\mathbf{q}} + \mathbf{K}\mathbf{q} = \mathbf{F}(t) \tag{1}$$

The p × p mass, damping, and stiffness matrices, M, C, and K, respectively, are constant, the force F depends on time t, and q is the set of generalized coordinates used to represent the configuration of the system.

A precursor of the HHT method is the Newmark method, in which a family of integration formulas that depend on two parameters β and γ is defined:

$$\mathbf{q}_{n+1} = \mathbf{q}_n + h\dot{\mathbf{q}}_n + \frac{h^2}{2}\left[(1-2\beta)\ddot{\mathbf{q}}_n + 2\beta\ddot{\mathbf{q}}_{n+1}\right] \tag{2a}$$

$$\dot{\mathbf{q}}_{n+1} = \dot{\mathbf{q}}_n + h\left[(1-\gamma)\ddot{\mathbf{q}}_n + \gamma\ddot{\mathbf{q}}_{n+1}\right] \tag{2b}$$

These formulas are used to discrete at time tn+1 the equations of motion (1) using an integration step size h:

$$\mathbf{M}\ddot{\mathbf{q}}_{n+1} + \mathbf{C}\dot{\mathbf{q}}_{n+1} + \mathbf{K}\mathbf{q}_{n+1} = \mathbf{F}_{n+1} \tag{2c}$$

Based on Eq. (2a) and (2b), q_{n+1} and $\dot{q}_{n+1}$ are functions of the acceleration $\ddot{q}_{n+1}$, which in Eq. (2c) remains the sole unknown quantity that is obtained as the solution of a linear system.

$$\gamma \geq 1/2 \qquad \beta \geq \frac{(\gamma + \frac{1}{2})^2}{4} \tag{3}$$

The only combination of β and γ that leads to a second-order integration formula is $\gamma = ½$ and $\beta = ¼$. This choice of parameters produces the trapezoidal method, which is both stable and second order. The drawback of the trapezoidal formula is that it does not induce any numerical damping in the solution, which makes it impractical for problems that have high-frequency oscillations that are of no interest or parasitic high-frequency oscillations that are a by-product of the finite element discretization process. Thus, the major drawback of the Newmark family of integrators was that it could not provide a formula that was stable and second order and displayed a desirable level of numerical damping.

The HHT method came as an improvement because it preserved the stability and numerical damping properties, while achieving second order accuracy when used in conjunction with the second order linear ODE problem of Eq. (1). The method proposed in actually does not change the expression of the Newmark integration formulas, but rather the form of the discretized equations of motion in (2c). The new equation in which the integration formulas of Eqs. (2a) and (2b) are substituted is

$$\mathrm{M}\ddot{\mathrm{q}}_{n+1} + (1+\alpha)\,\mathrm{C}\dot{\mathrm{q}}_{n+1} - \alpha\mathrm{C}\dot{\mathrm{q}}_n + (1+\alpha)\,\mathrm{K}\mathrm{q}_{n+1} - \alpha\mathrm{K}\mathrm{q}_n = \mathrm{F}\left(\tilde{t}_{n+1}\right) \tag{4}$$

where

$$\tilde{t}_{n+1} = t_n + (1+\alpha)\,h \tag{5}$$

The HHT method uses a single parameter called $\alpha \in [-1/3\ ,\ 0]$ and

$$\gamma = \frac{1-2\alpha}{2} \qquad \beta = \frac{(1-\alpha)^2}{4} \tag{6}$$

The smaller the value of α, the more damping is induced in the numerical solution. Note that in the limit, the choice $\alpha = 0$ leads to the trapezoidal method with no numerical damping.

Chapter 4

Results and discussion

4.1 General

After completing the modeling procedure for all mentioned type of truss bridges and time-history inputs, dynamic analysis is carried out for all types of bridge models and for all time-history inputs. Maximum displacement at centre of the bridge is taken as a primary response parameter for study. Firstly, response is obtained without tuned mass damper and then with one TMD system, three TMD system and finally with five TMD system.

4.2 Modal analysis of bridge

Modal analysis is carried out for all bridge models without TMD to get the fundamental frequency and time-period of all bridge (primary system) models. Following table 4.1 shows the same.

Table 4.1 Fundamental frequencies and time-periods of various bridge models

Bridge type	Fundamental Frequency (Hz) / Time Period (s)				
	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
45.7 m	**7.99** / 0.12	16.74 / 0.06	22.79 / 0.04	27.09 / 0.03	27.18 / 0.03
61.0 m	**5.24** / 0.19	11.21 / 0.08	14.25 / 0.07	16.02 / 0.06	16.05 / 0.06
76.2 m	**4.08** / 0.24	8.92 / 0.11	11.01 / 0.09	14.94 / 0.06	14.95 / 0.06

4.3 Behaviour of bridge under various TMD parameters

For understanding behaviour of response under variation of different TMD parameters, we will analyze 45.7 m bridge models with single TMD system under the Th101 and Th404 time-histories. This will form the basis of some fundamental facts.

- Fundamental frequency of 45.7 m bridge model = 7.99 Hz
- Dominating excitation frequency of Th101 time-history = 5.8 Hz, 4 Hz.
- Dominating excitation frequency of Th404 time-history = 6.8 Hz, 7.9 Hz.
- No. of TMD = 1
- Displacement at centre of bridge without TMD for Th101 = 26.16 mm
- Displacement at centre of bridge without TMD for Th404 = 25.46 mm
- Structural Damping = 2 %

4.3.1 Case 1: With constant TMD mass and varying TMD stiffness and zero TMD damping.

From the below table 4.2 and 4.3 in conjunction with fig. 4.1 and 4.2 , we can observe for both Th101 case and Th404 case that when TMD frequency is at 5.78 Hz and 6.45 Hz respectively, a frequency very close to the excitation frequency of the Th101 and Th404 input time-history, the displacement at centre of bridge is minimum (optimized). This has been observed with almost all time-histories. Thus it can be incurred that to get minimum response, the TMD frequency should be in a range (band) of dominating excitation frequency. This is in conjunction with the theory of tuned mass damper system.

Table 4.2 Sensitivity of response with respect to TMD frequency for Th101 (with constant mass)

Sensitivity of response (disp. at mid-span) w.r.t. TMD frequency with constant TMD mass and varying TMD stiffness and zero damping			
Frequency of TMD (Hz)	Mass of TMD (kg)	Stiffness of TMD (N/m)	Displacment at mid-span (mm)
10.00	5200	20507968	23.54
7.94	5200	12917591	20.68
7.69	5200	12134892	20.00
7.14	5200	10463249	20.39
6.25	5200	8010925	17.80
5.92	5200	7180410	16.90
5.88	5200	7096183	16.15
5.85	5200	7013429	15.57
5.81	5200	6932115	15.32
5.78	5200	6852206	14.69
5.71	5200	6699055	14.73
5.65	5200	6551037	14.95
4.76	5200	4650333	16.71
4.00	5200	3281275	19.24

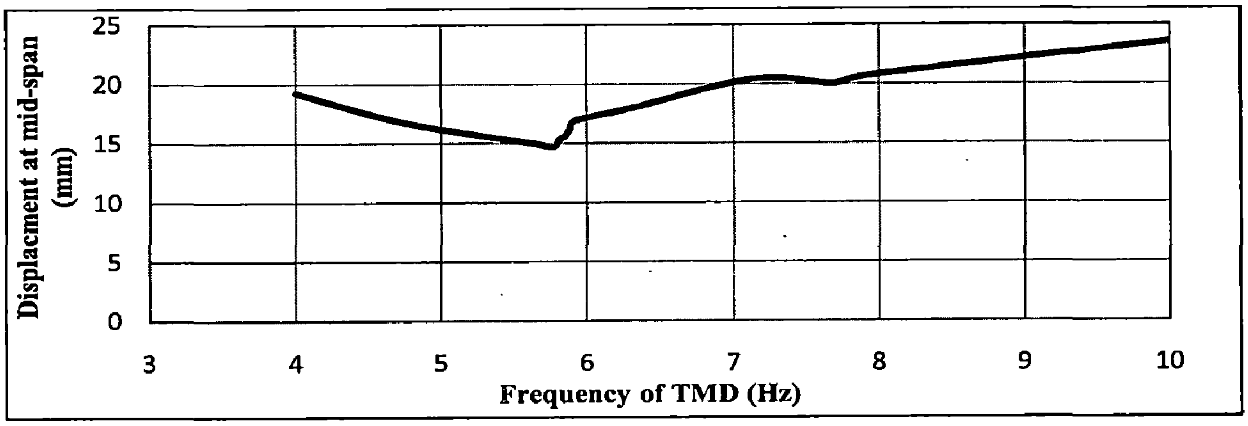


Figure 4.1 Graph showing response w.r.t. TMD frequency for Th101

Table 4.3 Sensitivity of response with respect to TMD frequency for Th404 (with constant mass)

Sensitivity of response (disp. at mid-span) w.r.t. TMD frequency for Th404 with constant TMD mass and varying TMD stiffness and zero TMD damping			
Frequency of TMD (Hz)	Mass of TMD (kg)	Stiffness of TMD (N/m)	Displacement at mid-span (mm)
8.33	5200	14241644	19.70
7.28	5800	12134892	19.90
7.14	5200	10463249	19.50
6.67	5200	9114652	16.60
6.45	5200	8536095	16.50
6.37	5200	8320000	16.80
6.25	5200	8010925	17.10
5.99	5200	7353425	18.50
5.95	5200	7266145	19.80
5.92	5200	7180410	20.10
5.75	5200	6773672	19.70
4.35	5200	3876743	20.90
4.17	5200	3560411	21.50
4.00	5200	3281275	21.90

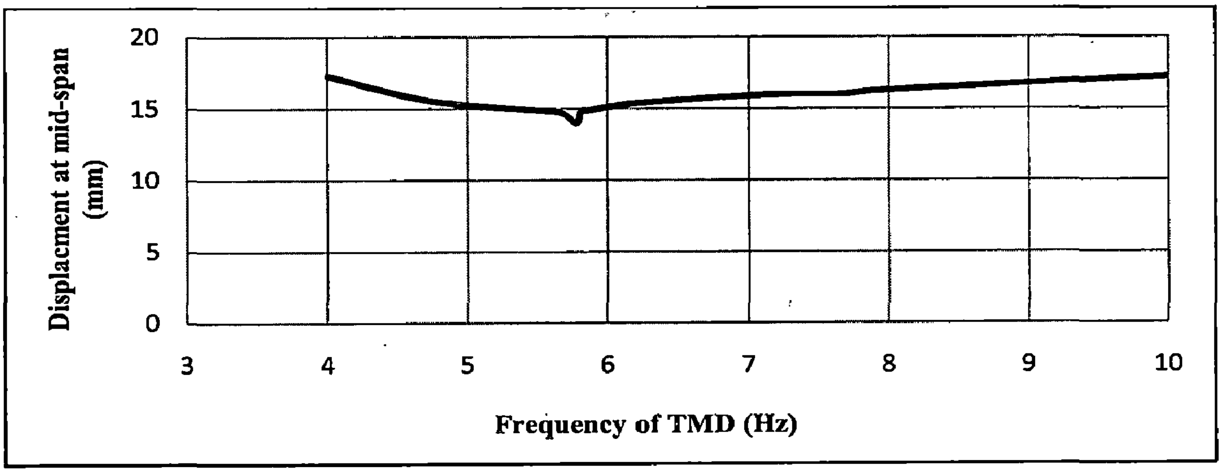


Figure 4.2 Graph showing response w.r.t. TMD frequency for Th404

4.3.2 Case 2: With varying TMD mass and constant TMD stiffness and zero TMD damping

Now we need to check that with frequency being same if we vary mass and keep stiffness constant, what effect it will create on the behavior of response. The analysis has been performed for Th101 as input time-history. Following table 4.4 and figure 4.3 shows that the optimum frequency still remains the same. This result shows that if we keep either mass constant and vary the stiffness or keep stiffness constant and vary the mass, it is the frequency of TMD that affects the response.

Table 4.4 Sensitivity of response with respect to TMD frequency for Th101 (with constant stiffness)

Sensitivity of response (disp. at mid-span) w.r.t. TMD frequency for Th101			
with varying TMD mass and constant TMD stiffness and zero TMD damping			
Frequency of TMD (Hz)	Mass of TMD (kg)	Stiffness of TMD (N/m)	Displacement at mid-span (mm)
10.00	1521	6000000	17.23
7.94	2415	6000000	16.24
7.69	2571	6000000	16.01
7.14	2982	6000000	15.94
6.25	3895	6000000	15.40
5.92	4345	6000000	14.95
5.88	4397	6000000	14.90
5.85	4449	6000000	14.85
5.81	4501	6000000	14.80
5.78	4553	6000000	13.95
5.71	4659	6000000	14.44
5.65	4766	6000000	14.75
4.76	6709	6000000	15.50
4.00	9508	6000000	17.25

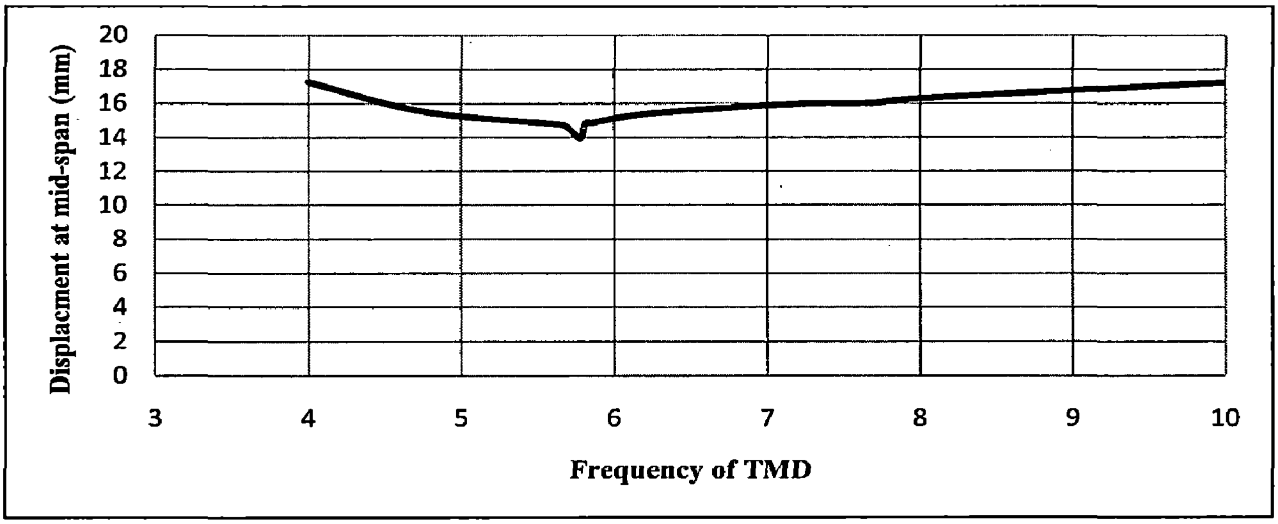


Figure 4.3 Graph showing response w.r.t. TMD frequency for Th101 (with constant stiffness)

4.3.3 Case 3: With varying TMD mass and varying TMD stiffness and zero TMD damping

Now both the mass and stiffness of TMD are varied to arrive at the same set of frequency for analysis for Th101 as input time-history. It is shown in table 4.5 and fig. 4.4. It was again observed that the optimum frequency (close to excitation frequency) that gives the minimum response still remains the same as in case 1 and case 2. Thus we can choose the value of TMD mass and stiffness (either of one) on practical basis to have desired frequency of TMD.

4.3.4 Case 4 : With constant TMD frequency i.e constant TMD mass and constant TMD stiffness and varying TMD damping.

Now after varying the basic frequency components of TMD i.e TMD mass and TMD stiffness, we will now determine the behavior of response under change in TMD damping. Following table 4.6 and fig.4.5 shows the effect on response of bridge at centre and response of TMD mass for different damping values of TMD. It is observed that as the TMD damping is increased the response of both bridge and TMD mass reduces. Variation is almost linear with the damping ratio of TMD. But there is always a practical imposition for the value of damping. Sadek et al *[11]* suggested 25 % damping ratio in TMD is good for the working of TMD. Damping may not be much effective in reducing the response of structure but is very much effective in reducing response of TMD mass (by 66 % as compared to 0 % damping) which is practically very important.

Table 4.5 Sensitivity of response with respect to TMD frequency for Th101 (with varying stiffness and varying mass)

Sensitivity of Response (disp. at mid-span) w.r.t. TMD frequency with varying TMD mass and varying TMD stiffness and zero damping			
Frequency of TMD (Hz)	Mass of TMD (kg)	Stiffness of TMD (N/m)	Displacement at mid-span (mm)
10.00	500	1971920	18.71
7.94	1000	2484152	16.49
7.69	1500	3500450	17.28
7.14	2000	4024327	16.90
6.25	2500	3851406	16.66
5.92	3000	4142544	16.15
5.88	3500	4776277	15.93
5.85	4000	5394945	15.62
5.81	4500	5998945	15.32
5.78	5000	6588660	14.19
5.71	5500	7082815	14.78
5.65	6000	7553079	15.01
4.76	6500	5812916	15.65
4.00	7000	4417101	18.12

Table 4.6 Sensitivity of response with respect to TMD frequency for Th101 (with varying stiffness and constant mass)

Sensitivity of Response (disp. at mid-span) w.r.t. TMD damping at same frequency						
with TMD mass constant and TMD stiffness constant and varying TMD damping						
Frequency of TMD (Hz)	Mass of TMD (kg)	Stiffness of TMD (N/m)	Effective Damping	Damping co-efficent (kg/s)	Displacment at mid-span (mm)	Displacement in TMD mass (mm)
5.78	5000	6588660	0.01	3630	13.9	61.6
5.78	5000	6588660	0.02	7260	13.6	56.0
5.78	5000	6588660	0.03	10890	13.4	52.7
5.78	5000	6588660	0.04	14520	13.1	49.0
5.78	5000	6588660	0.05	18150	12.9	45.3
5.78	5000	6588660	0.06	21780	12.6	41.6
5.78	5000	6588660	0.07	25410	12.4	37.9
5.78	5000	6588660	0.08	29040	12.2	39.0
5.78	5000	6588660	0.10	36300	11.7	35.6
5.78	5000	6588660	0.12	43560	11.0	32.0
5.78	5000	6588660	0.14	50820	10.9	30.9
5.78	5000	6588660	0.20	72601	9.9	25.6
4.75	5200	6588660	0.25	77558	9.5	23.3
7.75	5200	6588660	0.30	151850	8.9	19.9

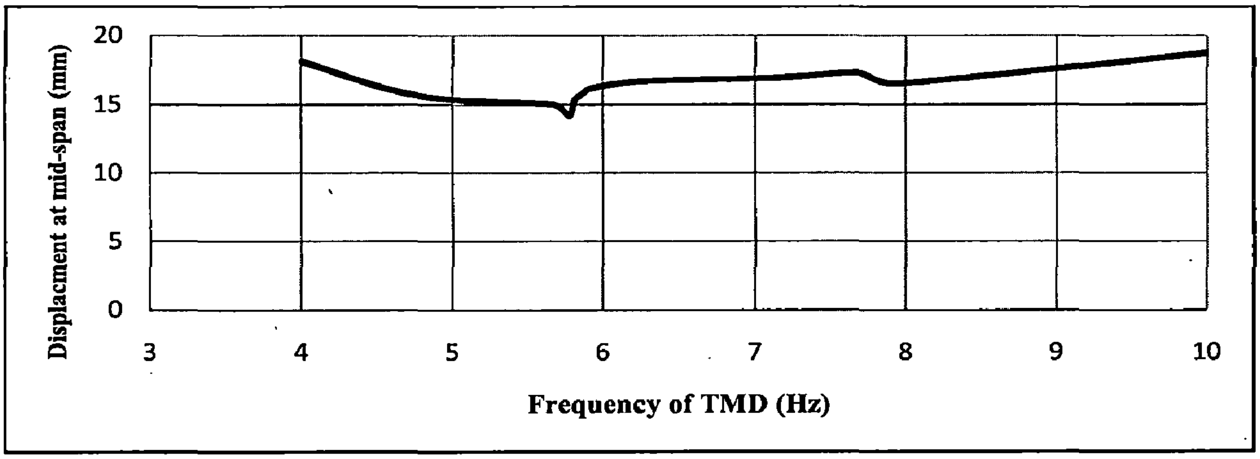


Figure 4.4 Graph showing response w.r.t. TMD frequency for Th101 (with varying stiffness and varying mass)

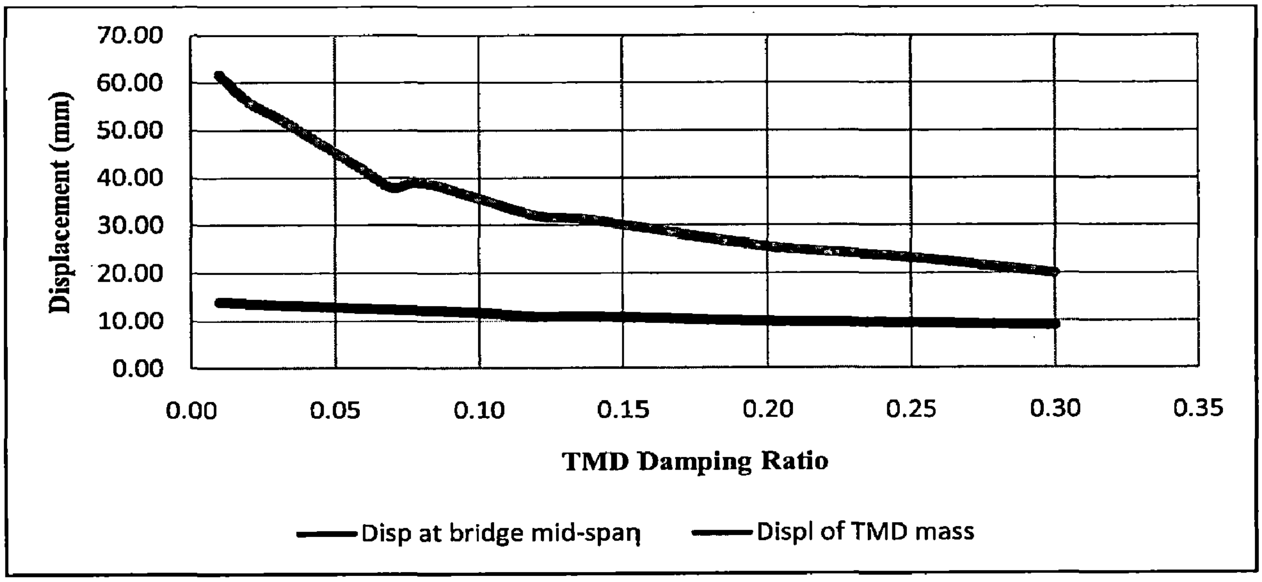


Figure 4.5 Graph showing displacement of bridge midspan and TMD mass for different TMD damping ratios

Thus from the above four cases it can be incurred that the frequency of TMD is the basic parameter for optimization of response and its mass and stiffness needs to be chosen based on practical conditions. Damping in TMD also holds quite a role in curbing down response of bridge and TMD mass (largely).

4.4 Efficiency of Single TMD over different time histories

Having formed the basis for analysis by observing the effect of various TMD parameters for single input time-history, let us now check its efficiency for other time-histories as well for the 45.7 m (same) model. Following table 4.7 shows dominating frequencies of various time-histories found using Fourier spectra of each time-history and the response of bridge for specific frequency of TMD (5.78 Hz) for 0% damping and 25 % damping. Table 4.7 also gives the percentage reduction of response for above cases with respect to response without TMD. Fig. 4.6 also shows the variation in graph form. We can see that damping in TMD has quite a effect on the response of bridge.

Further, table 4.8 shows the percentage response reduction for all time-histories for various frequencies of TMD with constant TMD damping of 25%. Also average response reduction for all time histories at different frequencies is calculated. It can be observed that we get the maximum reduction at f = 6.75 Hz. as major time-histories have got the dominating frequency range of 5.5 Hz to 7.5 Hz. Thus it can be said that the TMD with this particular frequency is able to get tune with most of the time-histories thus giving us the better result. But for single TMD it can be further optimized as there may be more optimized value.

Table 4.7 Displacement at bridge mid-span and TMD mass with and without TMD damping for all time-histories

Time-history										Displacment at mid-span (mm) for 45.7 m bridge model				
Name	Dominating Frequencies (Hz)									without TMD	with single TMD and varying damping			
											TMD mass (kg)	5200	TMD mass (kg)	5200
											TMD stiffness (N/m)	6851384	TMD stiffness (N/m)	6851384
											TMD frequency (Hz)	5.78	TMD frequency (Hz)	5.78
											TMD damping (kg/s)	0%	TMD damping (kg/s)	94381 (25%)
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10		Disp. at midspan	% Reduction	Disp. at midspan	% Reduction
Th101			4		5.8					26	14	48.29	9	64.97
Th202			3.7	4.3	5.7					24	15	38.02	9	60.47
Th303			4	5	5.9					25	19	22.12	9	62.87
Th404			3.7		5.2	6.8	7.9			31	19	36.80	9	71.86
Bageshwar			3.7		5.2				9.1	26	29	-10.35	13	48.32
Barkot			3.6			6.5				38	11	70.04	8	80.14
Chamoli	2		3.5							16	11	29.56	8	52.75
Champavat					5.9		7.8			34	25	27.78	12	63.73
Dhanaulti		3		4.7			7.2			19	16	12.34	11	40.52
Garsain				4.1		6.2	7			34	22	37.15	13	63.41
Ghanshali				4.7						27	18	34.00	10	63.06
Jubbal						6.5	7			27	14	48.09	10	63.84
Kapkal			4	4.5						17	14	19.26	10	43.33
Nathpa				4.1						13	17	-35.84	9	25.31
Pauri	2			4.8						24	15	36.50	8	68.07
Rampur		3	3.6							13	12	1.56	8	33.73
Rudraprayag					5.7	6.8				22	4	80.81	3	83.99
Tehri								8.2	9	20	17	15.08	9	53.89
Uttarkashi		2.5	3.2	4.2						10	18	-69.58	9	14.65
Average % reduction												23.24		55.73

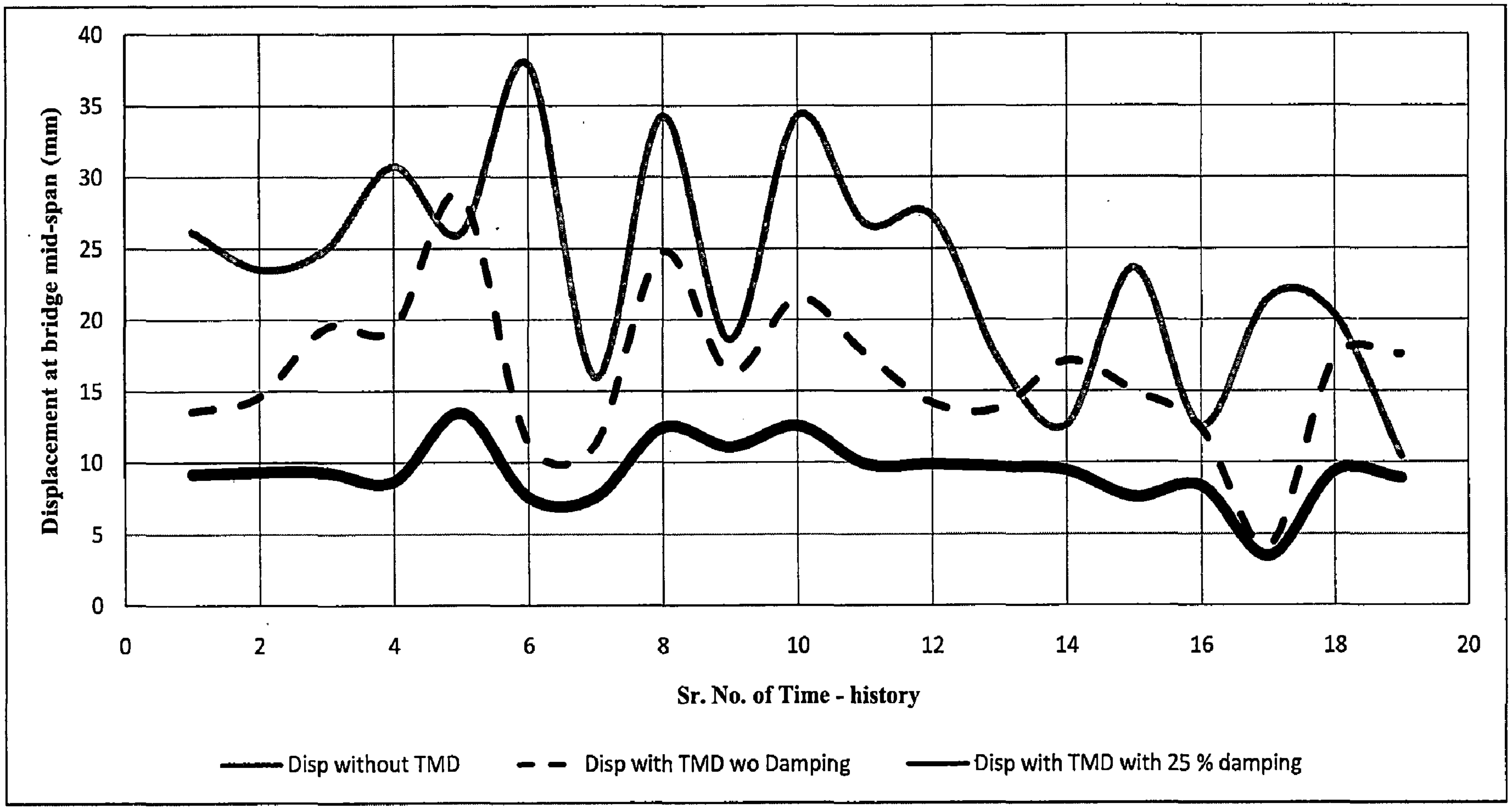


Fig. 4.6 Graph showing displacement at bridge mid-span and TMD mass with and without TMD damping for all time-histories.

Table 4.8 Displacement at bridge mid-span with 25 % TMD damping and different TMD frequency for all time-histories

Sr. No.	without TMD	Displacment at mid-span (mm)							
		with single TMD with 25 % TMD damping							
		TMD mass (kg)	5200	TMD mass (kg)	5200	TMD mass (kg)	5200	TMD mass (kg)	5200
		TMD stiffness (N/m)	6851384	TMD stiffness (N/m)	4627110	TMD stiffness (N/m)	9343943	TMD stiffness (N/m)	12317598
		TMD frequency (Hz)	5.78	TMD frequency (Hz)	4.75	TMD frequency (Hz)	6.75	TMD frequency (Hz)	7.75
		TMD damping (kg/s)	94381 (25 %)	TMD damping (kg/s)	77558 (25 %)	TMD damping (kg/s)	132256 (25 %)	TMD damping (kg/s)	151850 (25 %)
		Displacement at midspan	% Reduction	Displacement at midspan	% Reduction	Displacement at midspan	% Reduction	Displacement at midspan	% Reduction
1	26.1	9.1	64.97	11.3	56.59	6.7	74.25	7.8	70.15
2	23.5	9.2	60.47	9.5	59.20	7.5	68.05	7.4	68.52
3	24.9	9.2	62.87	11.6	53.31	8.0	67.78	9.3	62.71
4	30.7	8.6	71.86	11.3	62.98	8.3	72.88	9.6	68.56
5	26.0	13.4	48.32	16.4	36.93	10.2	60.68	10.0	61.24
6	37.9	7.5	80.14	8.8	76.56	7.9	79.13	10.1	73.34
7	15.9	7.5	52.75	9.2	42.10	7.2	54.72	9.0	43.61
8	34.2	12.4	63.73	15.7	53.95	9.0	73.56	10.5	69.08
9	18.6	11.0	40.52	14.2	23.40	8.3	55.11	10.1	45.25
10	34.2	12.5	63.41	14.1	58.81	10.3	69.71	10.6	68.93
11	26.7	9.8	63.06	11.3	57.74	8.7	67.38	10.7	59.67
12	27.2	9.8	63.84	13.2	51.36	8.7	67.71	11.1	59.24
13	17.0	9.6	43.33	9.6	43.47	9.1	46.11	9.2	45.80
14	12.6	9.4	25.31	11.3	10.25	8.3	33.83	8.7	30.59
15	23.6	7.5	68.07	9.7	58.66	6.1	73.82	6.9	70.58
16	12.5	8.3	33.73	10.0	19.92	7.3	41.36	8.3	33.35
17	21.5	3.4	83.99	3.1	85.36	3.8	82.31	4.2	80.43
18	20.2	9.3	53.89	11.8	41.74	7.4	63.45	9.0	55.24
19	10.3	8.8	14.65	11.5	-11.26	6.3	38.45	7.0	31.91
Avearge % reduction			55.73		46.37		62.65		57.80

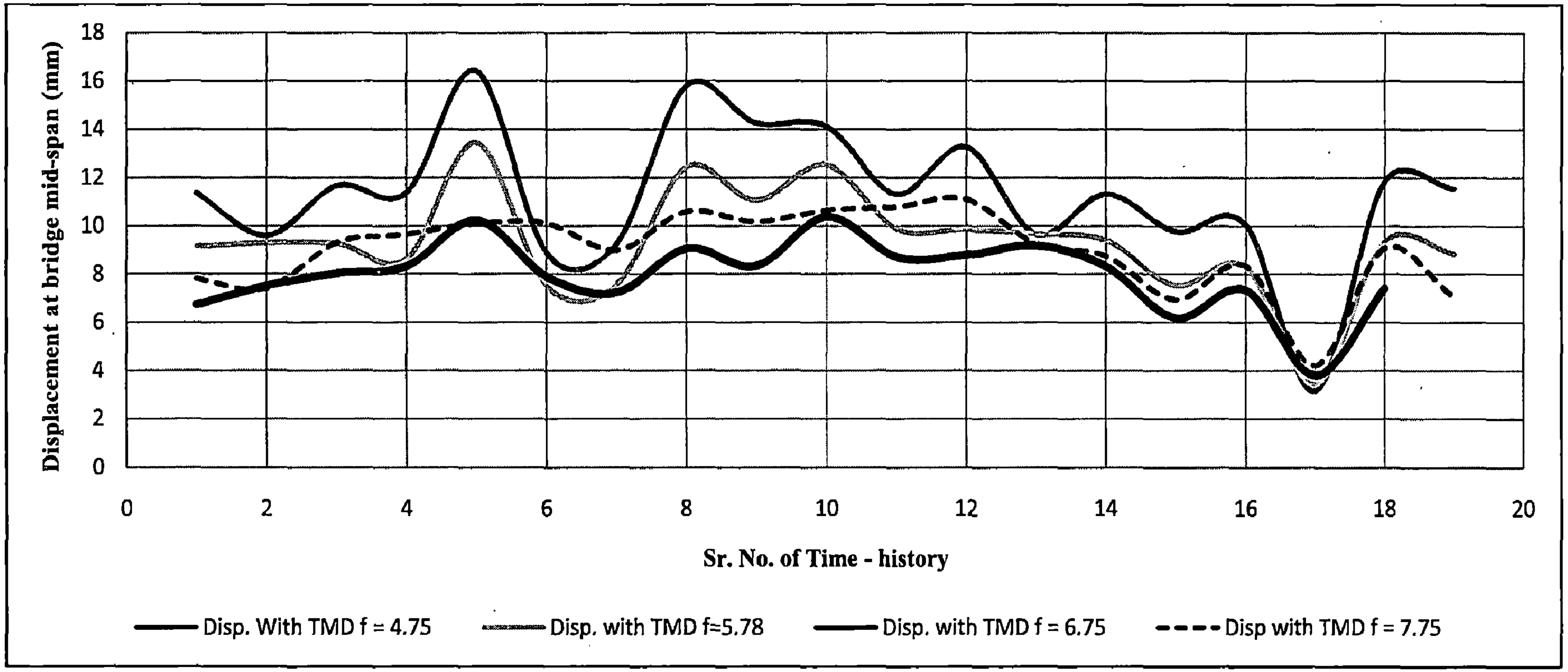


Figure 4.7 Graph showing displacement at bridge mid-span with 25 % TMD damping and different TMD frequency for all time-histories

4.5 Efficiency of 3 TMD System w.r.t shift in frequency band for 45.7 m bridge

Table 4.9 shows the average reduction of response over all time time-histories for 45.7 m bridge when applied with 3 TMD system with shift in TMD frequency band but with same band width. We can observe that there exists a frequency band which gives the best result. Fig 4.8 shows the reduction over different time-histories.

4.6 Efficiency of 3 TMD System w.r.t change in its frequency band-width for 45.7 m bridge

Table 4.10 shows the average reduction of response over all time time-histories for 45.7 m bridge when applied with 3 TMD system with change in frequency band with different TMD band widths. We can observe that there exist a frequency bandwidth which gives the best result.Thus the band width of TMD system should not be too broad nor too narrow. Fig 4.9 shows the reduction over different time-histories for the same.

Table 4.9 Displacement at mid-span with 3 TMD with 25 % damping with shift in TMD frequency band

Time-history											Displacement at mid-span (mm) with 3 TMD with 25 % damping (Comparison w.r.t shift in frequency)											
	Dominating Frequencies (Hz)									without TMD	TMD Sys 1	1	2	3	TMD Sys 2	1	2	3	TMD Sys 3	1	2	3
											TMD m(kg)	2000	2000	2000	TMD m(kg)	2000	2000	2000	TMD m(kg)	2000	2000	2000
											TMD k (N/m)	2607864	3593824	4737538	TMD k (N/m)	1779658	2607864	4737538	TMD k (N/m)	2839565	4737538	6039005
											TMD f (Hz)	5.75	6.75	7.75	TMD f (Hz)	4.75	5.75	7.75	TMD f (Hz)	6.75	7.75	8.75
											TMD c (kg/s)	36110	42390	48670	TMD c (kg/s)	29830	36110	48670	TMD c (kg/s)	37680	48670	54950
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10		Displacement (mm)		% Reduction		Displacement (mm)		% Reduction		Displacement (mm)		% Reduction	
1			4		5.8					26	6		75		8		71		9		67	
2			3.7	4.3	5.7					24	7		70		8		65		9		60	
3			4	5	5.9					25	8		69		8		68		10		61	
4			3.7		5.2	6.8	7.9			31	8		74		8		74		10		67	
5			3.8		5.3				9.2	26	9		64		12		55		12		54	
6			4			7				38	8		80		8		80		9		75	
7	2		4							16	7		55		7		57		9		43	
8					5.9		8			34	8		77		10		70		10		70	
9		3		5			7.3			19	8		57		9		51		10		46	
10				4		6.3	7			34	9		73		11		68		12		65	
11				5						27	8		69		9		68		10		61	
12						7	7			27	8		70		9		66		10		62	
13			4	5						17	9		48		9		50		11		35	
14				4						13	8		35		8		33		10		18	
15	2			5						24	6		75		6		74		8		66	
16		3	4							13	7		43		8		38		9		27	
17					5.8	6.8				22	4		83		4		83		6		73	
18								8	9	20	7		66		8		60		9		55	
19		2.5	3	4.3						10	6		44		7		28		8		20	
Average % reduction													65				61				54	

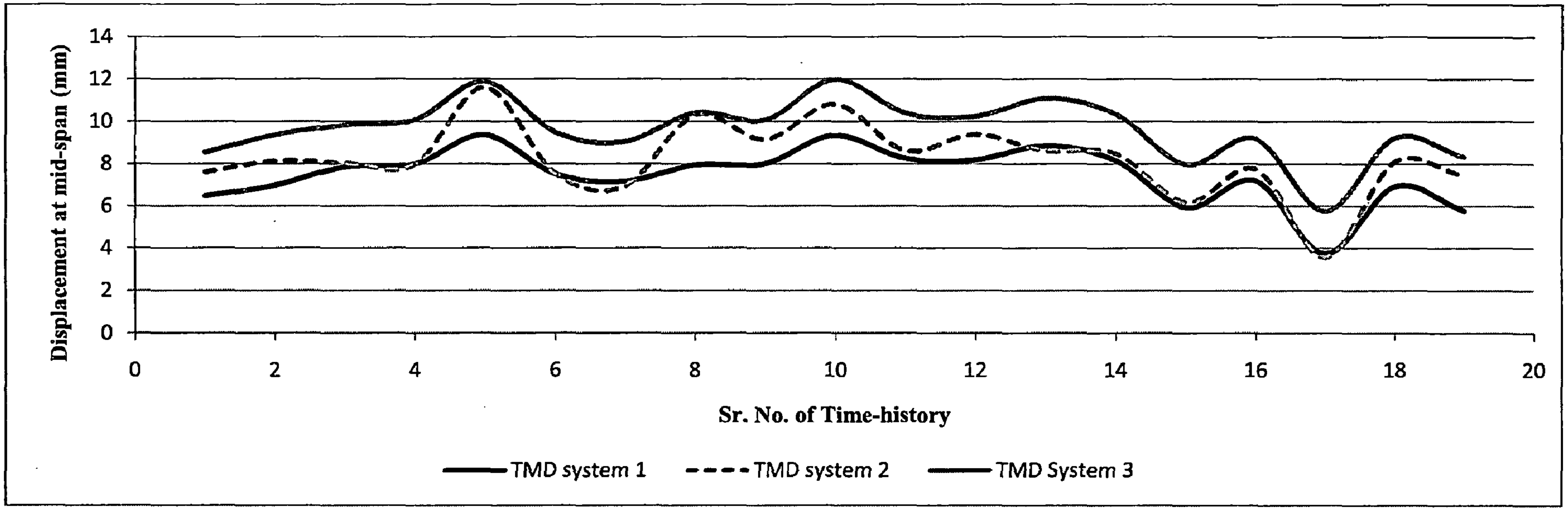


Figure 4.8 Graph showing displacement at mid-span with 3-TMD system with 25 % damping for different TMD frequency band

Table 4.10 Displacement at mid-span with 3-TMD system with 25 % damping with change in TMD frequency band-width

Time-history											Displacement at mid-span (mm) with 3 TMD with 25 % damping (Comparison w.r.t change in frequency band-width)											
Sr. No.	Dominating Frequencies (Hz)									without TMD	Narrow Frequency Range of TMD				Medium Frequency Range of TMD				Broader Frequency Range TMD			
											TMD Sys 1	1	2	3	TMD Sys 2	1	2	3	TMD Sys 3	1	2	3
											TMD m(kg)	2000	2000	2000	TMD m(kg)	2000	2000	2000	TMD m(kg)	2000	2000	2000
											TMD k (N/m)	3081125	3593824	4145962	TMD k (N/m)	2607864	3593824	4737538	TMD k (N/m)	1109205	3593824	6039005
											TMD f (Hz)	6.25	6.75	7.25	TMD f (Hz)	5.75	6.75	7.75	TMD f (Hz)	3.75	6.75	8.75
											TMD c (kg/s)	39250	42390	45530	TMD c (kg/s)	36110	42390	48670	TMD c (kg/s)	23550	42390	54950
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10		Displacement (mm)		% Reduction		Displacement (mm)		% Reduction		Displacement (mm)		% Reduction	
1			4		5.8					26	7		75		6		75		7		74	
2			3.7	4.3	5.7					24	7		69		7		70		7		69	
3			4	5	5.9					25	8		69		8		69		8		66	
4			3.7		5.2	6.8	7.9			31	8		74		8		74		8		73	
5			3.7		5.2				9.1	26	10		62		9		64		10		61	
6			3.6			6.5				38	7		80		8		80		8		78	
7	2		3.5							16	7		56		7		55		8		52	
8					5.9		7.8			34	8		75		8		77		10		72	
9		3		4.7			7.2			19	8		57		8		57		9		54	
10				4.1		6.2	7			34	10		71		9		73		9		74	
11				4.7						27	8		69		8		69		9		67	
12						6.5	7			27	8		70		8		70		10		64	
13			4	4.5						17	9		47		9		48		8		50	
14				4.1						13	8		34		8		35		8		39	
15	2			4.8						24	6		75		6		75		6		75	
16		3	3.6							13	7		43		7		43		8		37	
17					5.7	6.8				22	4		82		4		83		4		83	
18								8.2	9	20	7		64		7		66		8		62	
19		2.5	3.2	4.2						10	6		39		6		44		6		44	
Average % reduction													64				65				63	

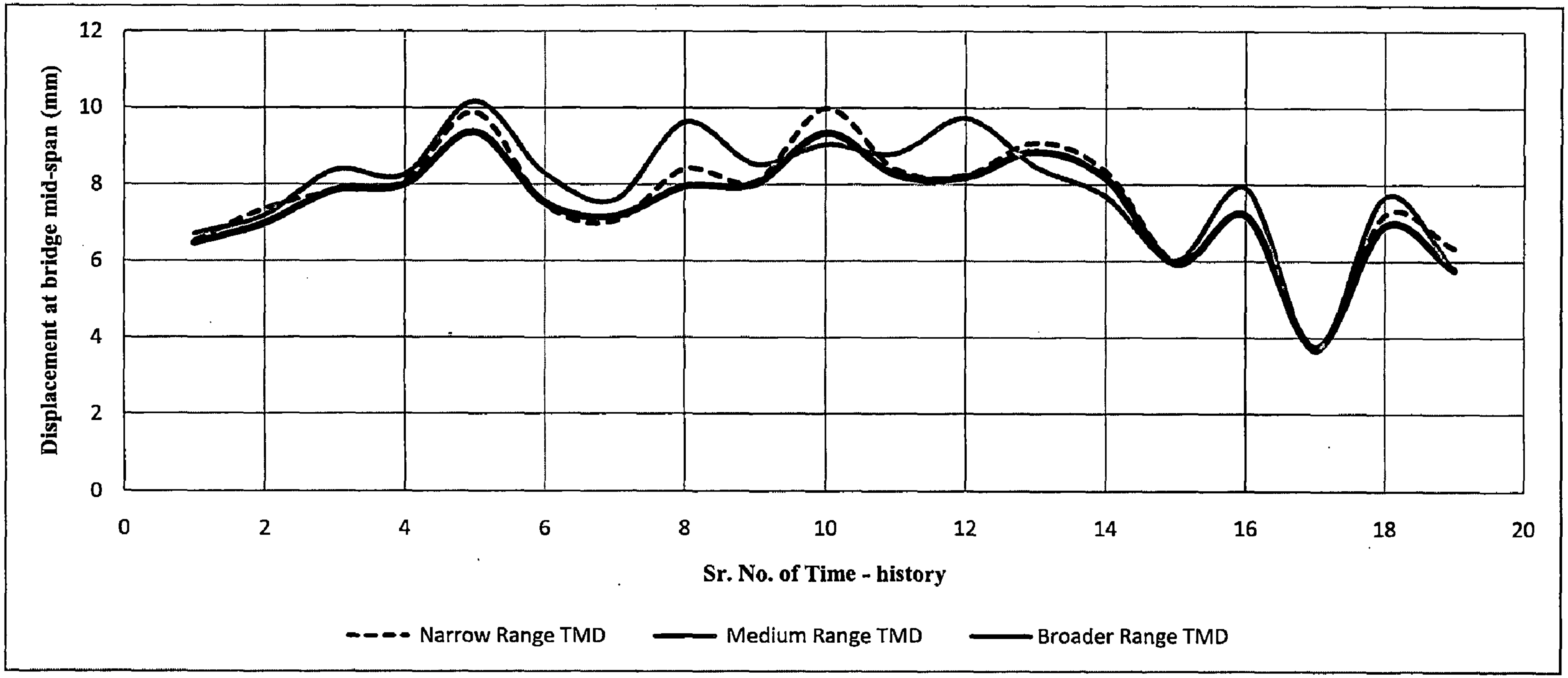


Fig 4.9 Graph showing displacement at mid-span with 3-TMD system with 25 % damping with change in TMD frequency band-width

4.7 Efficiency of 5 TMD System for different bridges:

4.7.1 45.7 m bridge

Now TMD system with 5 TMD is attached at the mid-span of bridge. Different frequency band-widths of TMD system were taken and it is found that TMD system was most effective when moderate size frequency band-width was used. Table 4.11 in conjunction with fig. 4.10 confirms the result. Also it can be observed that we get the maximum response reduction as compared to system with single TMD ssytem and 3 TMD system as 5 TMD system covers a wider range of tuning frequencies that get tune with the broad range of excitation time-histories.

4.7.2 61.0 m bridge

Different TMD frequency band-widths were taken and it was found that TMD system was most effective when moderate size frequency band-width was used. Table 4.12 in conjunction with fig. 4.11 confirms the result. More response reduction is obtained as fundamental frequency of bridge is near to the centre of TMD frequency band. Also this band covers most range of excitation frequencies. Thus this bridge gives the maximum reduction in response.

4.7.3 75.6 m bridge

Keeping frequency band-width of TMD system same, change was made in the central TMD frequency. It is found that response for each case was not very different. Thus it can be said that for TMD system, the small change in one of the frequency will not make much difference in the overall response. It is the frequency properties of total TMD system that governs the response. Table 4.13 in conjunction with fig. 4.12 confirms the result.

4.8 Comparison of displacement time-histories for all bridges

Displacement time-histories of mid-span point of 45.7 m, 61.0 m and 75.6 m bridge with and without TMD under Ghanshali earthquake time-history are shown in Figure 4.13, 4.14 and 4.15 respectively. 5 TMD system has been used for comparison. It can be seen that there is a huge reduction in the displacement along the whole time-history with TMD. Reduction in the overall response reduces the stresses in the structure and thus helps in reducing damage.

Table 4.11 Displacement at mid-span with 5 TMD system for various frequency band-width size for 45.7 m bridge model

Displacement at mid-span (mm) with 5 TMD with 25 % damping (Comparison w.r.t change in frequency band-width)

	Very Narrow Frequency Range of TMD						Medium Frequency Range of TMD						Broader Frequency Range TMD					
	TMD Sys 1	1	2	3	4	5	TMD Sys 2	1	2	3	4	5	TMD Sys 3	1	2	3	4	5
	TMD m(kg)	1200	1200	1200	1200	1200	TMD m(kg)	1200	1200	1200	1200	1200	TMD m(kg)	1200	1200	1200	1200	1200
	TMD k (N/m)	1564719	1703739	2156295	2487577	2842523	TMD k (N/m)	1067795	1564719	2156295	2842523	3623403	TMD k (N/m)	357903	1067795	2156295	3623403	5469120
	TMD f (Hz)	5.75	6	6.75	7.25	7.75	TMD f (Hz)	4.75	5.75	6.75	7.75	8.75	TMD f (Hz)	2.75	4.75	6.75	8.75	10.75
	TMD c (kg/s)	21666	22608	25434	27318	29202	TMD c (kg/s)	17898	21666	25434	29202	32970	TMD c (kg/s)	10362	17898	25434	32970	40506

Sr. No.	Time-history: Dominating Frequencies (Hz) 1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	without TMD	Very Narrow: Displacement (mm)	Very Narrow: % Reduction	Medium: Displacement (mm)	Medium: % Reduction	Broader: Displacement (mm)	Broader: % Reduction
1			4		5.8					26	6	76	6	76	7	74
2			3.7	4.3	5.7					24	7	71	7	71	7	69
3			4	4.5	5.9					25	8	69	8	69	8	66
4			3.7		5.2	6.8	7.9			31	8	74	8	74	8	73
5			3.7		5.2				9.1	26	9	64	9	66	10	61
6			3.6			6.5				38	7	81	8	80	8	78
7	1.5		3.5							16	7	56	7	55	8	52
8					5.9		7.8			34	8	77	8	76	10	72
9		3		4.7			7.2			19	8	58	8	58	9	54
10				4.1		6.2	7			34	9	73	8	75	9	74
11				4.7						27	8	70	8	70	9	67
12						6.5	7			27	8	71	8	70	10	64
13			4	4.5						17	9	49	8	51	8	50
14				4.1						13	8	36	8	39	8	39
15	2			4.8						24	6	75	6	76	6	75
16		3	3.6							13	7	43	7	41	8	37
17					5.7	6.8				22	4	83	4	83	4	83
18								8.2	9	20	7	66	7	67	8	62
19		2.5	3.2	4.2						10	6	44	5	50	6	44
Average % reduction												65		66		63

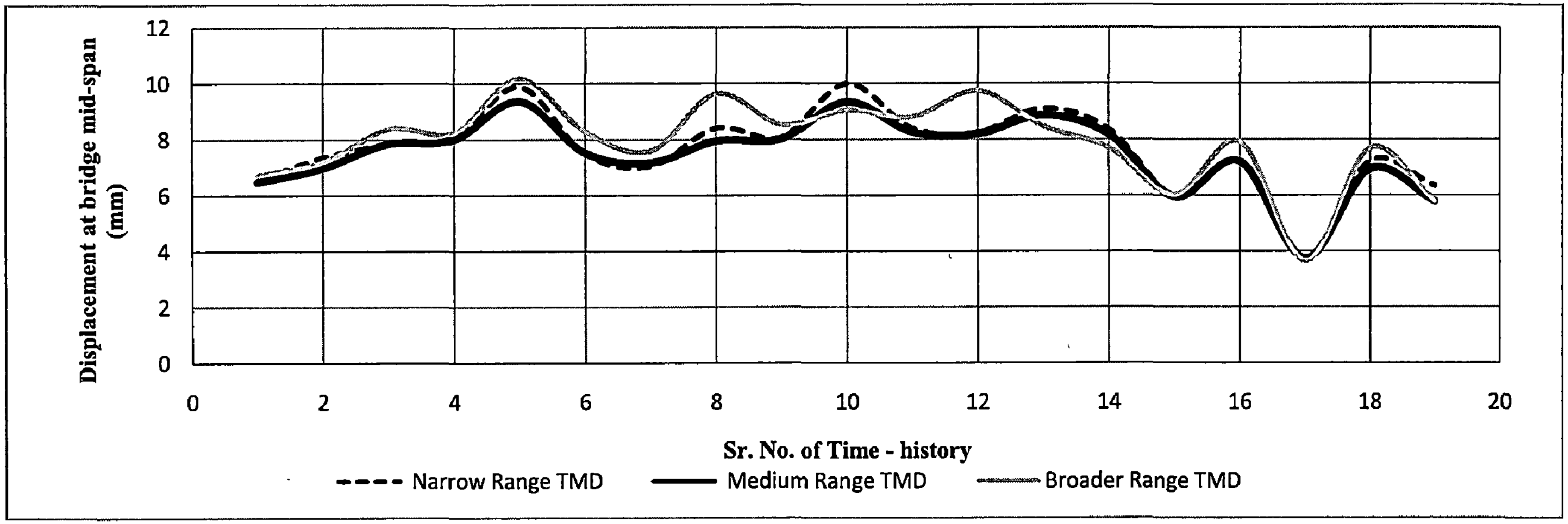


Figure 4.10 Graph showing displacement at mid-span with 5 TMD system for various frequency band-width size for 45.7 m bridge model

Table 4.12 Displacement at mid-span with 5 TMD system for various frequency band-width size for 61.0 m bridge model

Time-history											Displacement at mid-span (mm) with 5 TMD with 25 % damping for 61 m bridge model																		
Sr. No.	Dominating Frequencies (Hz)									without TMD	Very Narrow Frequency Range of TMD						Medium Frequency Range of TMD						Broader Frequency Range TMD						
											TMD Sys 1	1	2	3	4	5	TMD Sys 2	1	2	3	4	5	TMD Sys 3	1	2	3	4	5
											TMD m(kg)	1800	1800	1800	1800	1800	TMD m(kg)	1800	1800	1800	1800	1800	TMD m(kg)	1800	1800	1800	1800	1800
											TMD k (N/m)	2347078	2555608	3234442	3731366	4263784	TMD k (N/m)	1601692	2347078	3234442	4263784	5435105	TMD k (N/m)	536855	1601692	3234442	5435105	8203680
											TMD f (Hz)	5.75	6	6.75	7.25	7.75	TMD f (Hz)	4.75	5.75	6.75	7.75	8.75	TMD f (Hz)	2.75	4.75	6.75	8.75	10.75
											TMD c (kg/s)	32499	33912	38151	40977	43803	TMD c (kg/s)	26847	32499	38151	43803	49455	TMD c (kg/s)	15543	26847	38151	49455	60759
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10		Displacement at midspan (mm)		% Reduction				Displacement at midspan (mm)		% Reduction				Displacement at midspan (mm)		% Reduction				
1			4		5.8					41	20		52				7		84				18		56				
2			3.7	4.3	5.7					42	20		52				8		82				17		60				
3			4	5	5.9					52	19		64				9		83				18		66				
4			3.7		5.2	6.8	7.9			55	22		59				8		85				20		64				
5			3.8		5.3				9.2	43	20		55				11		74				18		59				
6			4			7				55	18		67				8		85				21		62				
7	2		4							33	18		47				8		77				18		46				
8					5.9		8			46	17		62				11		76				15		68				
9		3		5			7.3			56	29		48				10		83				22		61				
10				4		6.3	7			52	22		58				9		83				19		63				
11				5						74	29		61				9		88				21		71				
12						7	7			24	13		47				11		56				14		43				
13			4	5						42	36		16				9		80				28		35				
14				4						62	37		41				8		86				28		54				
15	2			5						34	18		46				6		82				14		57				
16		3	4							25	17		34				8		67				17		33				
17					5.8	6.8				14	6		58				3		76				6		59				
18								8	9	42	17		60				8		81				16		62				
19		2.5	3	4.3						20	14		33				6		69				12		43				
Average % reduction													51						79						56				

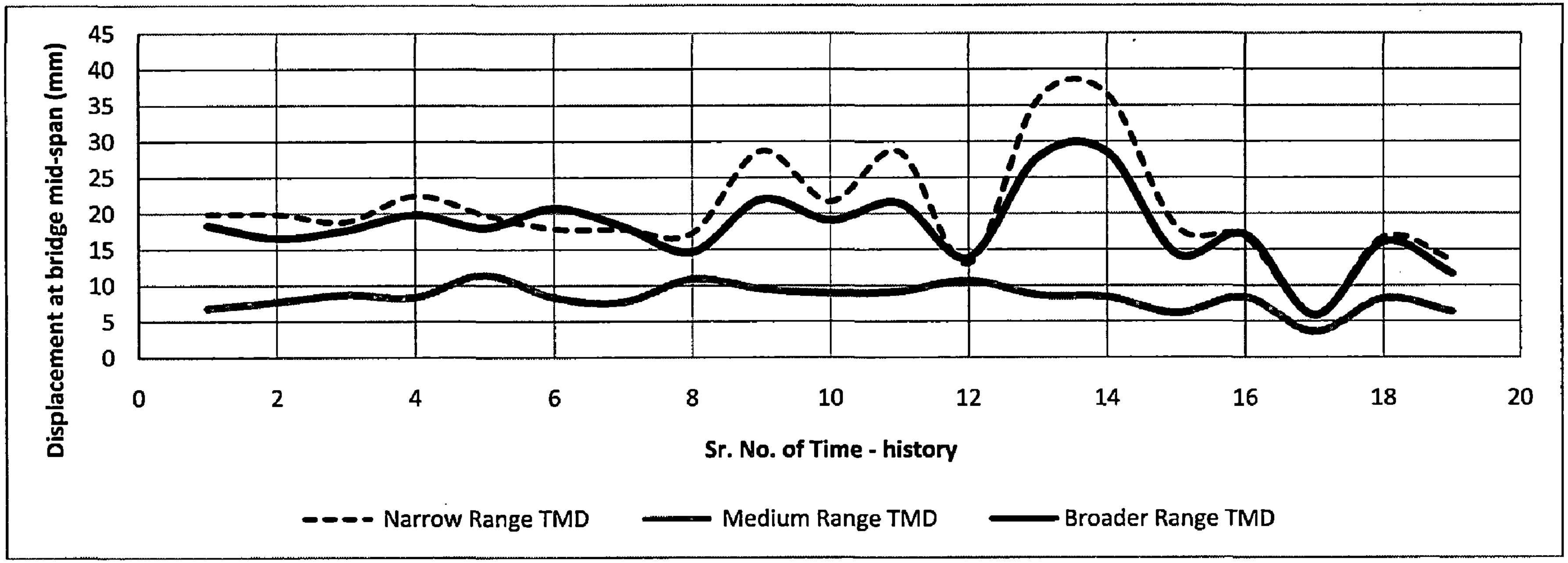


Figure 4.11 Graph showing displacement at mid-span with 5 TMD system for various frequency band-width size for 61.0 m bridge model.

Table 4.13 Displacement at mid-span with 5 TMD system for different central TMD frequency but same frequency band-width size for 75.6 m bridge model

Time-history											Displacement at mid-span (mm) with 5 TMD with 25 % damping for 75.6 m bridge model																	
Sr. No.	Dominating Frequencies (Hz)									without TMD	Broader Frequency Range TMD - High Central TMD frequency						Broader Frequency Range TMD - Medium Central TMD frequency						Broader Frequency Range TMD - Low Central TMD frequency					
											TMD Sys 1	1	2	3	4	5	TMD Sys 2	1	2	3	4	5	TMD Sys 3	1	2	3	4	5
											TMD m(kg)	2500	2500	2500	2500	2500	TMD m(kg)	2500	2500	2500	2500	2500	TMD m(kg)	2500	2500	2500	2500	2500
											TMD k (N/m)	745632	1386506	2224572	3259830	4492280	TMD k (N/m)	745632	1386506	1780890	3259830	4492280	TMD k (N/m)	745632	1386506	1577536	3259830	4492280
											TMD f (Hz)	2.75	3.75	4.75	5.75	6.75	TMD f (Hz)	2.75	3.75	4.25	5.75	6.75	TMD f (Hz)	2.75	3.75	4	5.75	6.75
											TMD c (kg/s)	21588	29438	37288	45138	52988	TMD c (kg/s)	21588	29438	33363	45138	52988	TMD c (kg/s)	21588	29438	31400	45138	52988
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10		Displacement (mm)		% Reduction				Displacement (mm)		% Reduction				Displacement (mm)		% Reduction			
1			4		5.8					82	21		74				19		77				20		76			
2			3.7	4.3	5.7					62	21		65				20		67				21		67			
3			4	5	5.9					75	22		70				21		72				21		72			
4			3.7		5.2	6.8	7.9			66	24		64				22		66				23		65			
5			3.8		5.3				9.2	49	25		49				24		52				24		50			
6			4			7				63	22		65				20		68				20		68			
7	2		4							46	23		50				21		54				22		53			
8					5.9		8			43	17		61				15		64				16		63			
9		3		5			7.3			66	33		50				30		55				31		53			
10				4		6.3	7			77	20		74				18		77				19		76			
11				5						58	21		63				20		66				20		65			
12						7	7			22	10		54				10		57				10		55			
13			4	5						129	37		71				33		75				34		73			
14				4						134	35		74				31		77				33		76			
15	2			5						36	15		60				13		63				14		62			
16		3	4							69	34		51				33		52				33		52			
17					5.8	6.8				17	7		60				7		63				6		63			
18								8	9	81	25		69				23		72				23		71			
19		2.5	3	4.3						43	17		59				16		63				16		63			
Average % reduction													62						65						64			

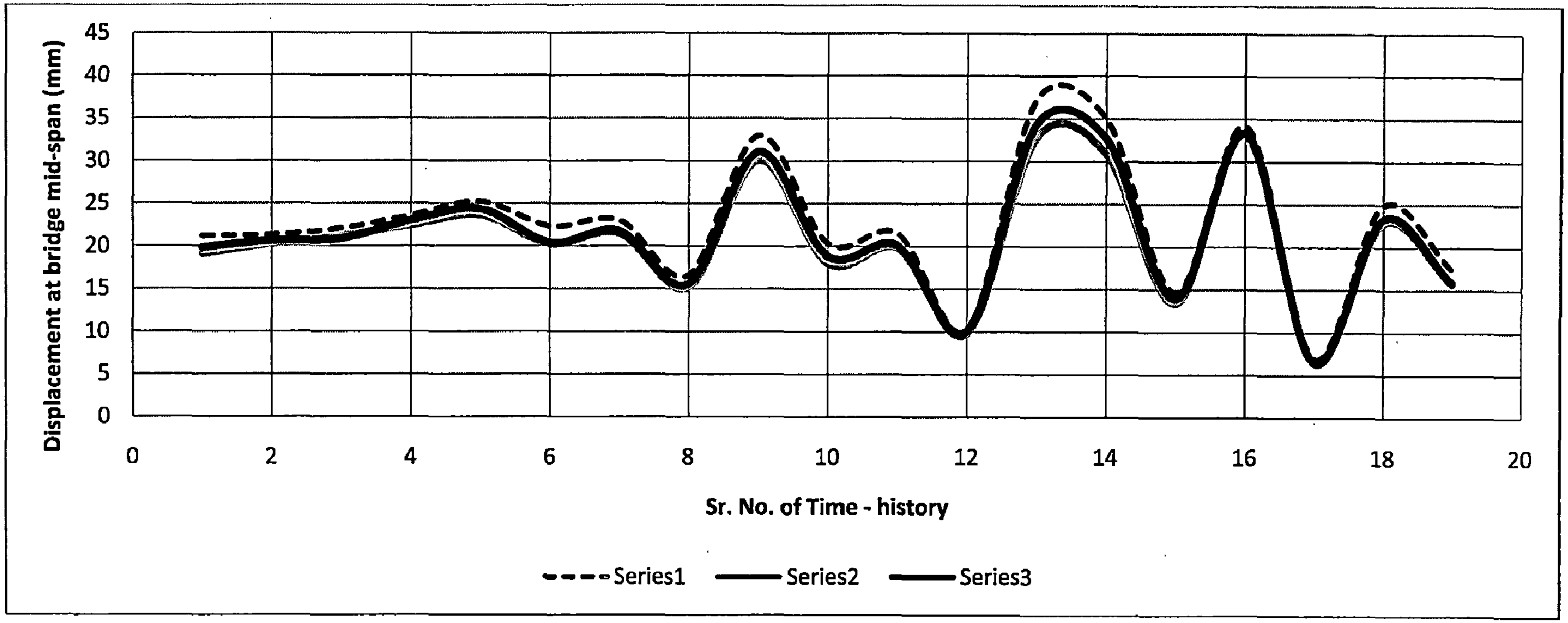


Figure 4.12 Graph showing displacement at mid-span with 5 TMD system for different central TMD frequency but same frequency band-width size for 75.6 m bridge model

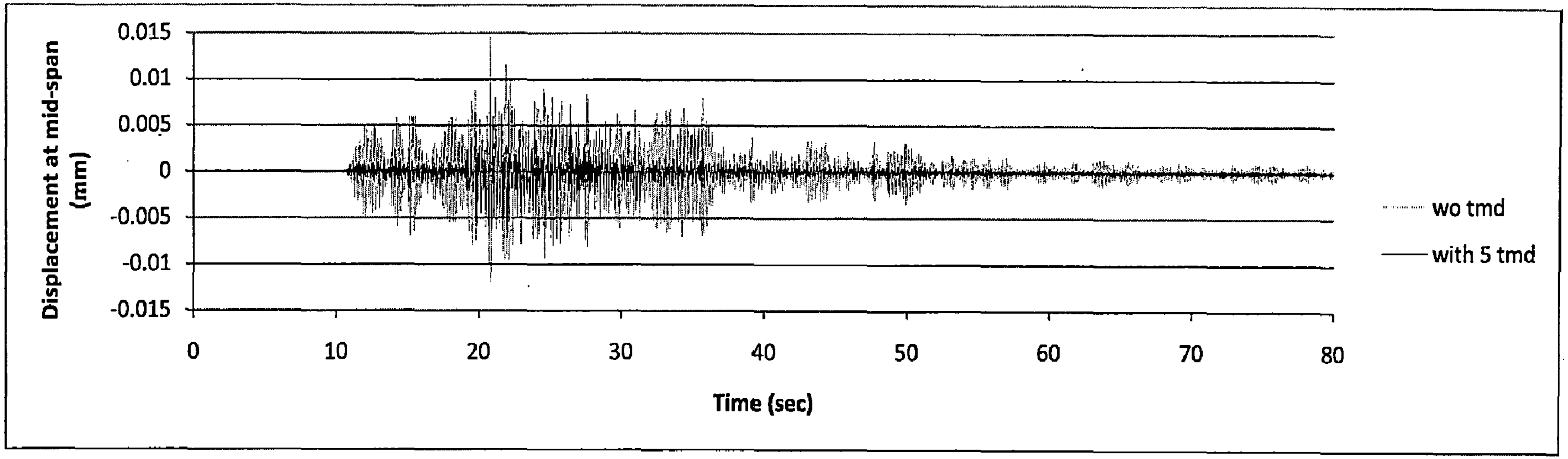


Figure: 4.13 Comparison between displacement histories of mid-span for Ghanshali input with and without 5 TMD for 45.7 m bridge.

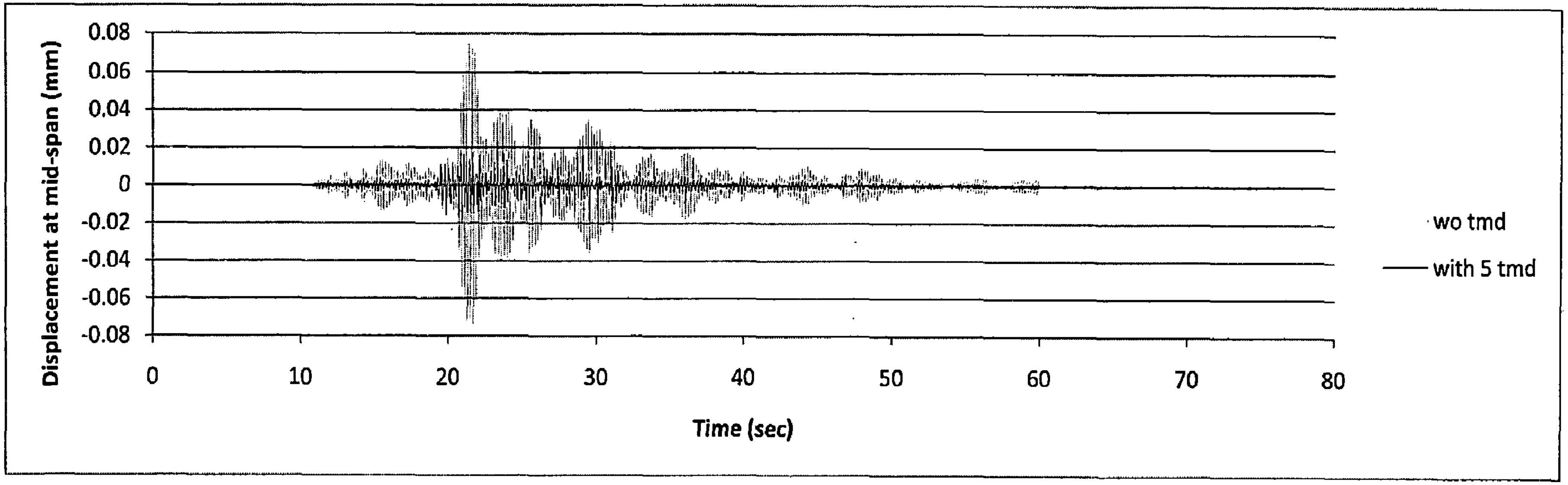


Figure: 4.14 Comparison between displacement histories of mid-span for Ghanshali input with and without 5 TMD for 61.0 m bridge.

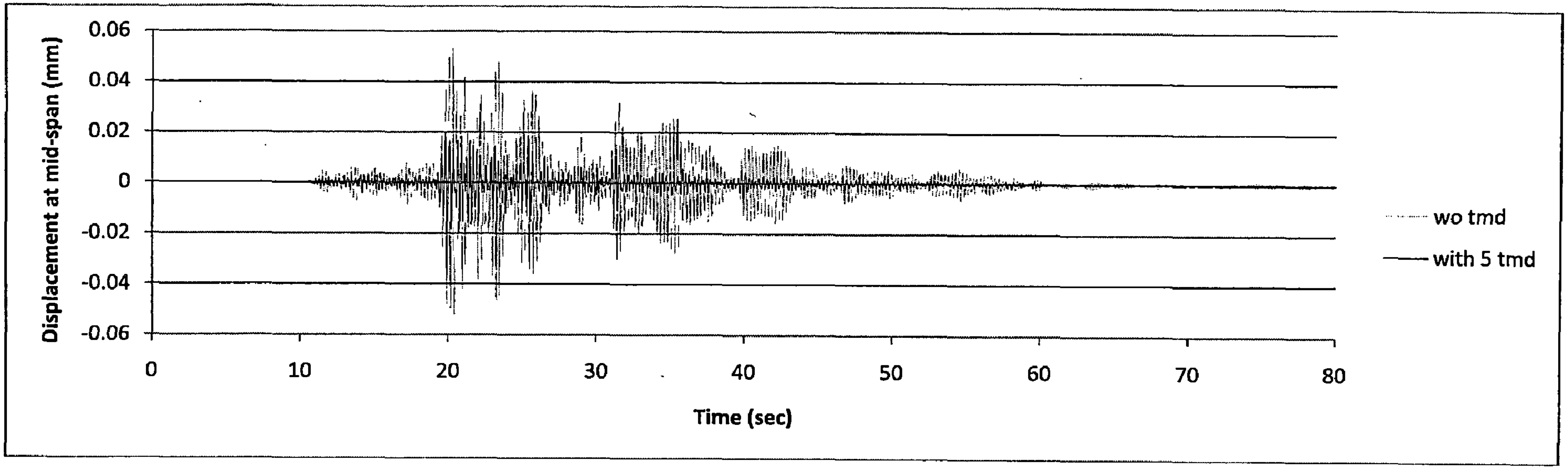


Figure: 4.15 Comparison between displacement histories of mid-span for Ghanshali input with and without 5 TMD for 75.6 m bridge.

Chapter 5

Summary and Conclusions

Tuned mass damper system is proposed for reduction in vertical response of bridge under vertical vibrations of earthquake. Three steel truss bridges are taken for study. Earthquake time-histories of Uttarakhand region are used as seismic laoding inputs.

Summary

The aim of the thesis is to check the efficacy of tuned mass damper system on steel truss bridges to reduce its vertical response under vertical vibrations of earthquake. First, three steel truss railway bridges with span 45.7 m, 61.0 m and 75.6 m were selected for study and their structural details were discussed. 3-D analytical models of all bridges were developed and modal analysis was performed to get their dynamic properties. 2-D analysis was performed in the plane of longitudinal and vertical axis of bridge. Fifteen earthquake time-history record of Uttarakhand region were taken as loading time-history input. Thereafter, response spectrum was calculated for each time-history. An average response spectrum is calculated from these response spectra. Four spectrum compatible time-histories were then generated from average response spectra. So total of nineteen time-histories was used in the analysis. Vertical displacement at the mid-span of bridge was taken as response parameter for study. Linear direct time-history analysis is used for the study. Initially, response was calcuted for each bridge under each time-history without TMD. Thereafter, all bridges were analysed with single TMD, three TMD and five TMD system with varying frequency attributes of TMD. Comparison is made for different cases and following conclusions were derived.

Conclusions

Analysis of different type of bridges with span *45.7 m*, *61.0 m* and *75.6 m* was performed with no TMD, single TMD, 3 TMDs and 5 TMDs. Following conclusions can be drawn from the analysis results:

- TMD is effective in reducing the vertical response of bridge under earthquake given that it is close to the excitation frequency. Mass and stiffness parameters of TMD do not affect the response provided that the frequency remains same.

- 2. Single TMD will be effective to very narrow range of excitation frequencies. In some excitation time-history cases the same configuration of TMD may increase the response. So to cater for broad range of excitation frequencies, it is better to have multiple TMD system.

- Frequency of TMD should be very close to excitation frequency range to get the maximun reduction in response

- For multiple TMD system, neither too narrow band-width nor too broad band-width gives the minimum response. It is the moderate frequency band-width of TMD that gives the best result.

- For multiple TMD system, particular band-width which covers the range of excitation frequencies gives the best result.

- Keeping frequency band-width of TMD system same, change is made in the central TMD frequency. It is found that response for each case was not very different. Thus it can be said that for TMD system, the small change in one of the frequency will not make much difference in the overall response. It is the frequency properties of total TMD system that governs the response.

- Increase in number of TMDs may not bring much improvement in reduction of response but decreases the risk of increase in response due to TMD for some earthquake time-histories.

- TMD system reduces the response over whole duration of time-history. This leads to reduction in stresses by large amount.

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