

## International Journal of Innovative Research in Science, Engineering and Technology

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# Behaviour of Steel Frame Structure under Dynamic Loading

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**ABSTRACT:** A research aims to study the behaviour of gable stay steel frame structures designed as per Indian codes under strong ground motions and dynamic wind load. For this a nonlinear time history analysis of the gable stay steel frame structure under various ground motions are carried using SAP2000 software. It is assumed that the structure is located at various regions of India and behaviour of those structures are compared. It is observed that storey drift of structure located under seismic zone III and below will be in the permissible limit and requires simple bracings but the structure located in seismic zone V will exceed the limit and Indian seismic code for design of steel structures are not enough to satisfy the design requirements.

**KEYWORDS:** Gable stay steel frame, Time history analysis, and Ground motions.

## I. INTRODUCTION

Earthquake are perhaps the most unpredictable and devastating of all natural disasters. They not only cause great destruction in terms of human casualties, but also have a tremendous economic impact on the affected area. Earthquake creates vibration on the ground that are translated into dynamic loads which causes the ground and anything attached to it to vibrate in a complex manner and cause damage to buildings and other structures. According to the recent Indian standard code on earthquake resistant design of structure more than 60-65% of the area of our country falls under seismic zone III and above. This underlines the importance of seismic detailing. For Indian site condition severe earthquakes have an extremely low probability of occurrence during the life of a structure. If a structure has to resist such earthquake elastically, it would require an expansive lateral load resisting system. So Indian seismic design code is for, "To ensure elastic behavior under a moderate earthquake which has a return period equal to the life of the structure and prevent collapse under the extreme probable earthquake."

Wind speeds vary randomly both in time and space and hence assessment of wind loads and response predictions are very important in the design of several buildings and structures. A large majority of structures met with in practice do not however, suffer wind induced oscillations and generally do not require to be examined for the dynamic effects of wind. For such normal, short and heavy structures, estimation of loads using static wind analysis has proved to be satisfactory. The details of this method involving important wind characteristics such as the basic wind speeds, terrain categories, modification factors, wind pressure and force coefficients, etc. Wind is air in motion relative to the surface of the earth. The primary cause of wind is traced to earth's rotation and differences in terrestrial radiation. The radiation effects are primarily responsible for convection either upwards or downwards.

At present the steel moment resisting frames are most widely used as the primary lateral load resisting system for seismic resistant steel building. This type of construction was considered as the safest one for earthquake resistance, as steel elements are expected to be able to sustain large plastic deformation in bending and shear. But the strength and ductility of MRF depends on Beam- Column connection. So Beam- Column connections are required to (a) remain elastic and not undergo inelastic or brittle mode failure in the connection elements and (b) resist the maximum probable

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beam moment and corresponding shear expected during strong seismic shaking when plastic hinges are formed in the beam. These two requirements are in the line with the strong column weak beam philosophy and use the capacity design concept.

The recent version of the code IS 800-2007 contains provision for design and detailing for seismic loads, but it does not suggest the type of connections which are suitable for high or intermediate seismic zones. The American institute of steel construction (AISC) has stipulated that connections in moment resisting frame should be qualified for use by testing. However, as testing of connection is time consuming and expensive, it has also specified a few pre-qualified connections which have been found to be satisfactory (FEMA 355D). These connections may be adopted in India also for better performance in strong or intermediate Earthquake.

Nonlinear time history analysis is an important technique for structural seismic analysis especially when the evaluated structural response is nonlinear. To perform such an analysis, a representative earthquake time history is required for a structure being evaluated. Time history analysis is a step by step analysis of the dynamic response of a structure to a specified loading that may vary with time. Time history analysis is used to determine the seismic response of a structure under dynamic loading of representative earthquake.

## II. RELATED WORK

**Wang and Wen (1994)**, investigate the “Evaluation of pre-Northridge low-rise steel buildings I: modeling.” Author developed a smooth connection-fracture hysteresis model, to account for the effects of flexible floor systems, biaxial interaction, and torsional motion, a 3D inelastic structural analysis 2 story model is used. Once connection fracture results in complete structural member failures, the response would increase drastically that collapse might occur. Therefore, it is concluded that fractured connections with some residual strength, even as low as 30% of the original, may act as energy dissipaters and have mitigating effects in structural response. On the other hand, once connecting fracture results in complete beam failures, the response is amplified considerably. It is also shown that total column failure further increases the maximum response and permanent displacement, confirming the expectation that column failure can cause collapse.

**Dhande, et al., (2000)**, presented the paper on “Industrial building design on seismic issues.” They researched to propose simple but innovative and effective lateral load resisting structural system (LLRSS) or structural technology and methodology for the seismic control which can be used in new as well as old structures. The author generated industrial building with braced frame model using SAP 2000; and analyzed with a SRSS method. The lateral load used for the analysis are Earthquake motion of Bhuj Earthquake as per IS1893-2002. They concluded that time period for braced frame will be less compared to unbraced frames. Braced model reduces the roof displacement significantly as compared to unbraced model. It also reduces the axial force, shear force, and bending moment in the column significantly compared to unbraced model.

**Paul, et al. (2000)**, had made a review on the “State of the art review on seismic design of steel moment resisting frames part II: strength and drift criteria.” The author explained about the seismic design provision related to strength and drift criteria as given in American and Indian code. American code adopted the strong column weak beam concept of frame design which is important in ensuring ductile behavior of the frame. The Indian code have no such specification. In American codes clear guidelines are available regarding the importance of including joint deformation is estimating the drift of a moment resisting frame. so drift estimation procedure must be clearly outlined in the Indian code.

## III. EXPERIMENTAL RESULTS

At present study behavior of 3D single bay industrial steel structure under strong ground motion located at various regions of India is investigated. The dimensions of the gable stay steel frame adopted for the nonlinear analysis is shown in Fig 1 and Table 1. The location of the structure assumed for the research are as follows,

- Mangalore, Karnataka which comes under the seismic zone III as per IS1893 (part 1): 2016 and wind zone II as per IS875 (part 3): 2015.

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- Darbhanga, Bihar which comes under the seismic zone V as per IS1893 (part 1): 2016 and wind zone VI as per IS875 (part 3): 2015.

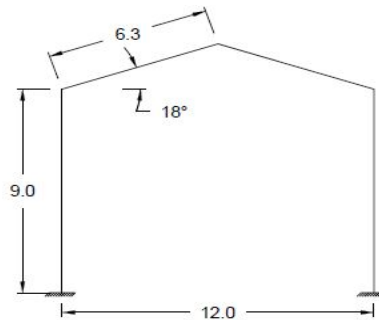


Fig 1. 2DGable stay frame model

Table 1. The general description of steel structures

Length	30m
Breadth	12m
Height of frame	11m
Number of frames	6nos
Yield strength of steel	250mpa
Grid to grid spacing in X direction	6m
Grid to grid spacing in Y direction	6m

The east coast and Gujrat on west coast of India is relatively more vulnerable for occurrences of cyclones. The structures located in this region are severely affected by the wind force. As per literature the wind loads are predominant while design of the members of steel structure so the coefficients used for the calculation of wind load acting on the structure located at both the regions are shown in table 2. The Indian site condition are experienced with an extreme earthquake having a magnitude of 7 and above. The earthquake causes both vertical and horizontal vibration to the structure. Though it has less effect on the individual members of steel frame but it leads to total storey drift of structure. This necessitates the seismic design of structure and the coefficients used for calculation of seismic load are shown in table 3. The final loads acting on the frame calculated based on location are shown in table 4.

Table 2. Wind coefficients [IS875 (Part 3): 2015]

Coefficients	Mangalore	Darbhanga
Basic wind speed, $V_b$	39m/s	55m/s
Risk coefficient, $K_1$	1	1
Terrain, height and structural size factor, $K_2$	0.8	1.05
Topography factor, $K_3$	1	1
Importance factor for cyclonic region, $K_4$	1.15	1.15
Design wind speed, $V_z$	35.88m/s	35.88m/s
Wind directionality factor, $K_d$	1	1
Area averaging factor, $K_a$	0.84	0.84
Combination factor, $K_c$	0.9	0.9
Design wind pressure, $P_z$	$577N/m^2 \geq 541N/m^2$ ( $0.7P_z$ )	$577N/m^2 \geq 541N/m^2$ ( $0.7P_z$ )

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Table 3. Seismic coefficients [IS1893 (Part 1): 2016]

Coefficients	Mangalore	Darbhanga
Zone factor, Z	0.16	0.36
Importance factor, I	1.2	1.2
Response reduction factor, R	3	3
Soil type	II medium stiff soil	II medium stiff soil

Table 4. Loads acting on the frame structure

Coefficients	Mangalore	Darbhanga
Dead load [IS875 (Part1): 1987]	0.27kN/m <sup>2</sup>	0.27kN/m <sup>2</sup>
Imposed load [IS875 (Part2): 1987]	0.59kN/m <sup>2</sup>	0.59kN/m <sup>2</sup>
Wind load on Roof [IS875 (Part3): 2015]	0.75kN/m <sup>2</sup>	1.664kN/m <sup>2</sup>
Wind load on Column [IS875 (Part3): 2015]	0.70kN/m <sup>2</sup>	1.56kN/m <sup>2</sup>
Seismic load [IS1893 (Part1): 2016]	3.19kN/m <sup>2</sup>	5.21kN

➤ **Model – 1**

The mathematical model of 2D gable stay frame is generated in SAP2000 software as shown in Fig 2. The load acting on the frame were assigned and model is analyzed for different load combination. The maximum moments, shear force and axial load diagrams obtained for different load combinations for individual members were noted and are shown in Table 5. The individual members were designed for the maximum moments, shear force and axial load and checked for deflection as per IS800: 2007. The final sizes of the members were shown in Table 6.

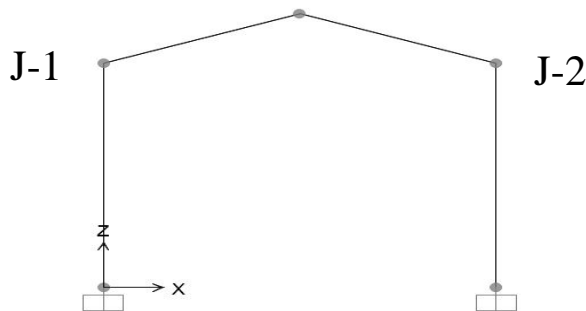


Fig 2. 2D gable stay steel frame SAP model

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Table 5. Axial force, shear force and bending moment diagrams under various load combinations

Load combination	Forces	Mangalore	Darbhanga
1.5(Dead load + Live load)	AFD		
	SFD		
	BMD		
1.5(Dead load + Wind load)	AFD		
	SFD		

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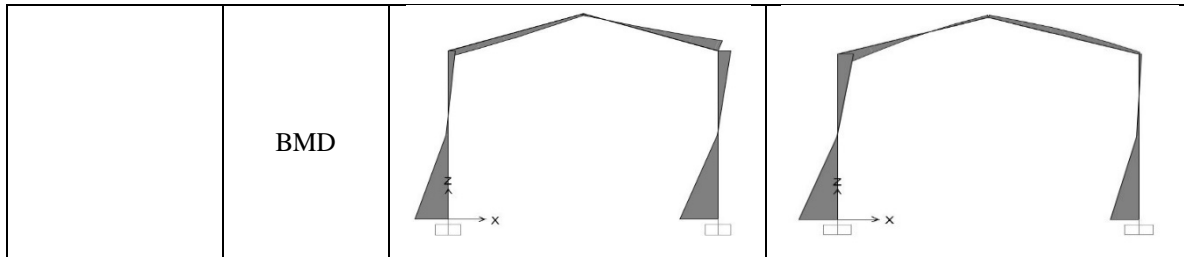


Table 6. Sectional properties of steel frame

Member	Mangalore	Darbhanga
Purlin and Runner	ISM150	ISM150
Beam	ISMB250	ISMB400
Column	ISMB400	ISMB550

### ➤ Model - 2

The 3D mathematical model of gable stay frame structure is generated in SAP2000 as shown in fig 3. The loads acting on the structure are defined and assigned to an individual members. The nonlinear time history analysis of an industrial steel frame is carried out under ground motions as mentioned in table 7. The plastic hinges are generated for column and beam member of a mathematical model as per ASCE 41-13. The ground motions are scaled as per spectral matching using time domain for peak ground acceleration of specific zone. The graph showing scaled pseudo spectral acceleration is shown in fig 4.

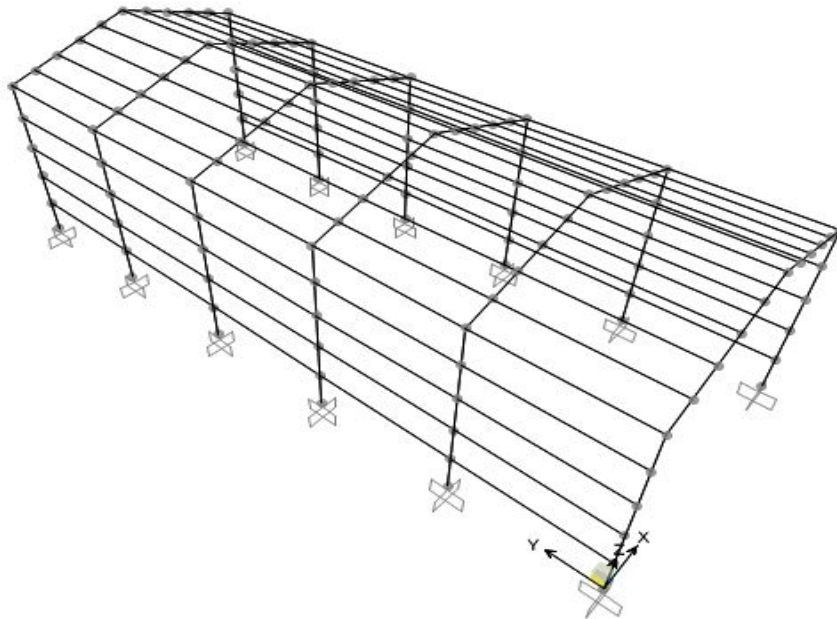


Fig 3. 3D gable frame structure model using SAP2000

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Table 7. Ground motions used for nonlinear time history analysis

Name	Station	Occurred	Magnitude	Points	Intervals
San Fernando	Pacoima dam	Feb 9, 1971	6.5	2086	0.02
Northridge	Sylmar county	Jan 17, 1994	6.7	3000	0.02
Loma Preita	Oakland outer harbor wharf	Oct 17, 1989	6.9	2000	0.02
Kern County	Taft Lincoln tunnel	July 21, 1952	7.3	2719	0.02
Imperial Valley	El Centro	May 18, 1940	6.5	2688	0.02

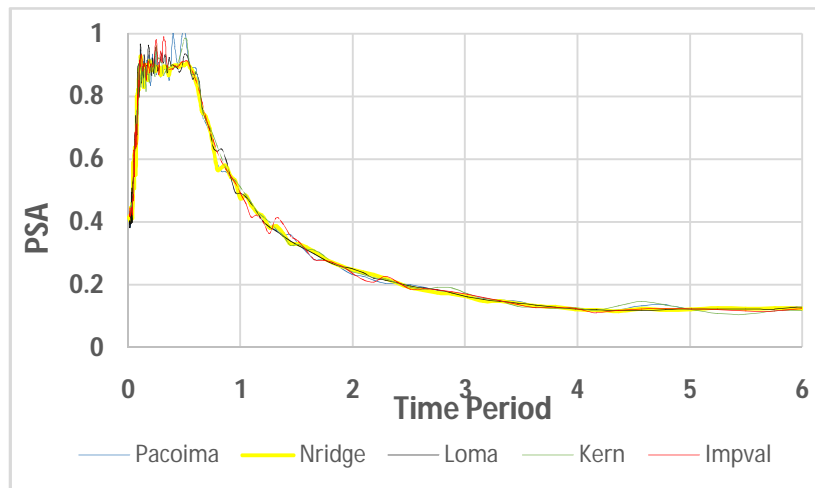
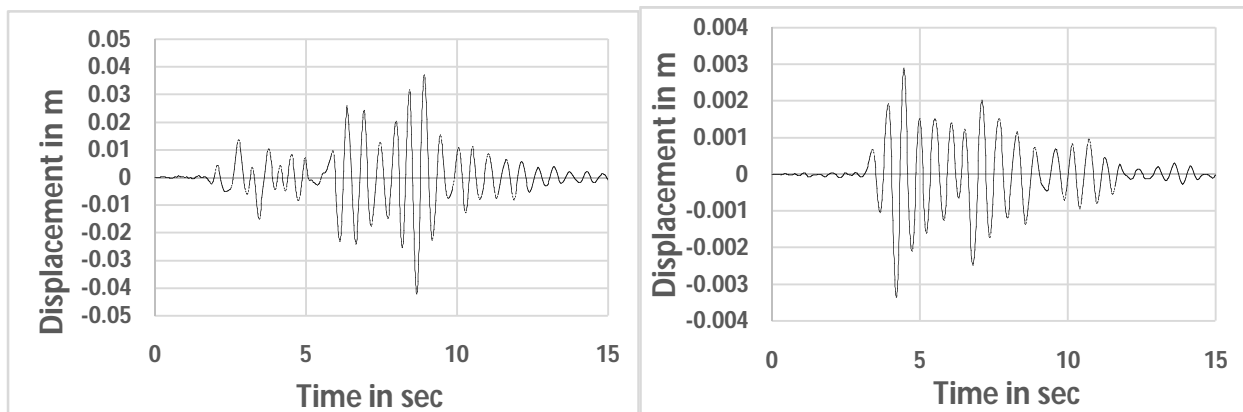


Fig 4. 3D gable frame structure model using SAP2000

After the nonlinear time history analysis of the 3D frame structure under the various scaled ground motions the displacement of the structure along lateral directions with respect to time at joint of column J-1 is extracted, a graph is plotted against displacement in m vs time period in sec for the structure located in Mangalore and Darbhanga region as shown in Fig 5 & 6 respectively.





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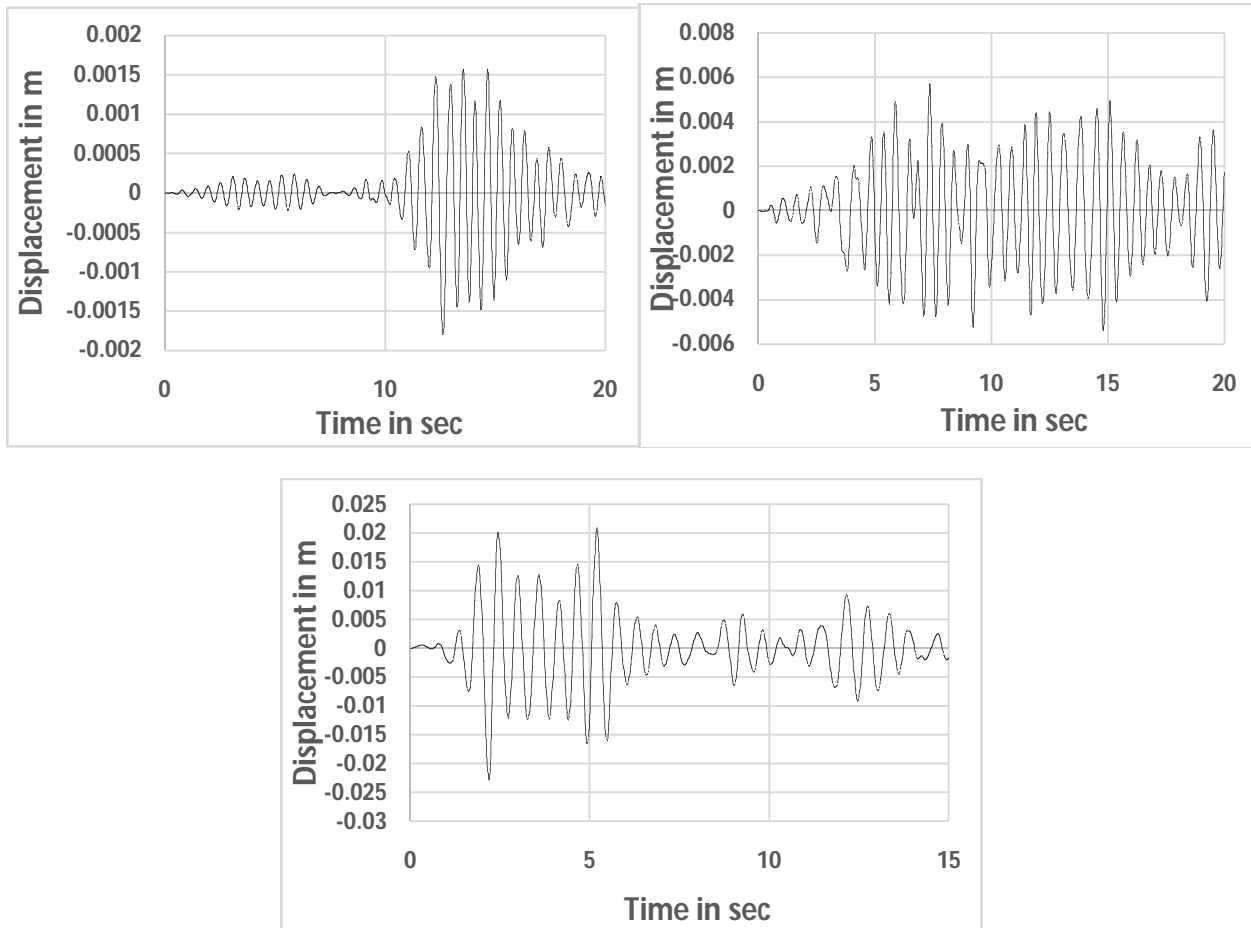
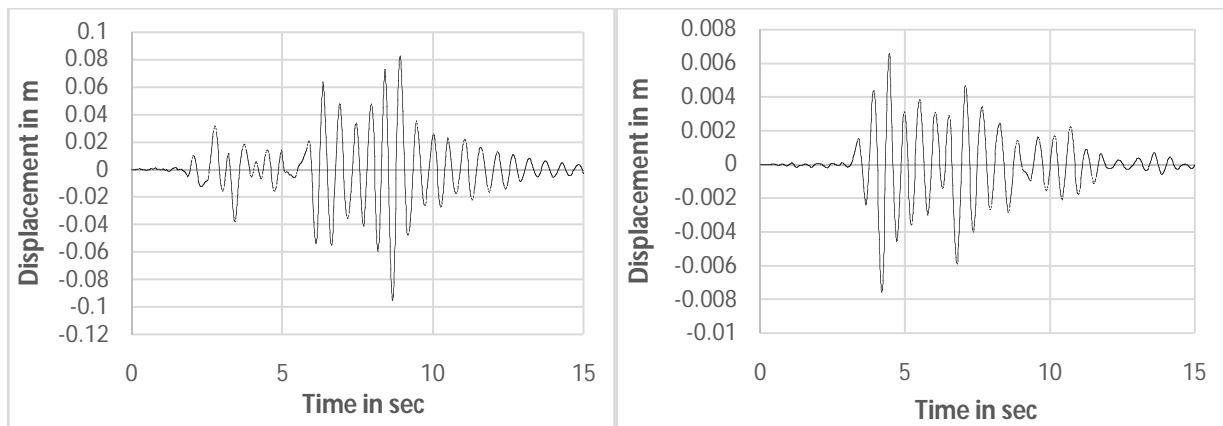


Fig 5. Displacement of gable frame structure located at Mangalore along lateral direction





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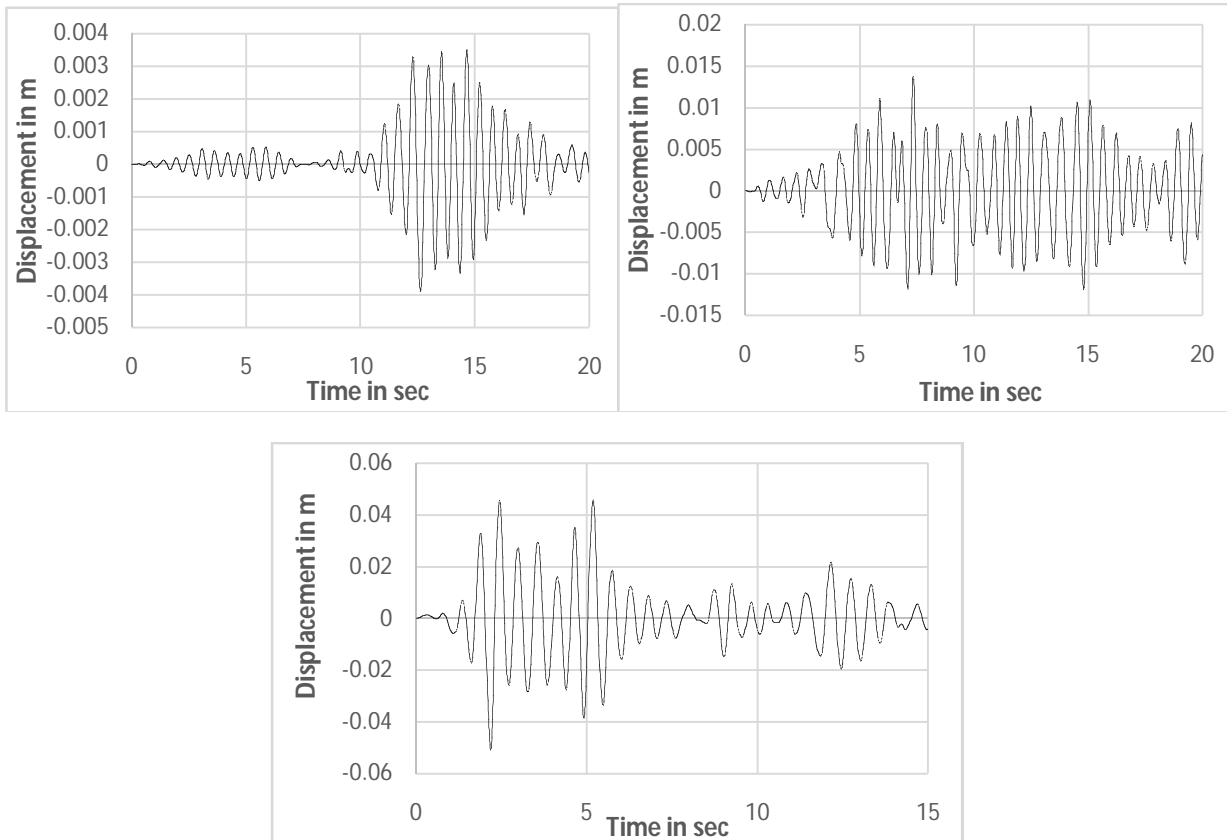


Fig 6. Displacement of gable frame structure located at Darbhanga along lateral direction

The displacement response spectra curve is generated by considering 5% damping at column joint J-1 for gable stay frame structure located at Mangalore and Darbhanga regions along lateral direction as shown in Fig 7& 8 respectively.

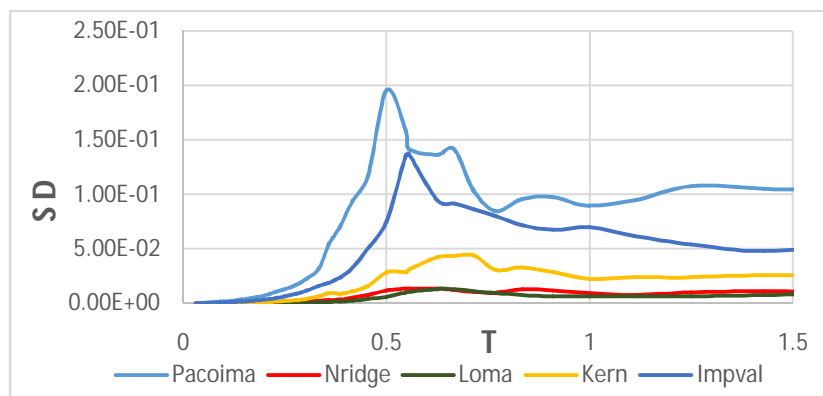


Fig 7. Displacement response spectra curve under 5% damping for structure at Mangalore region

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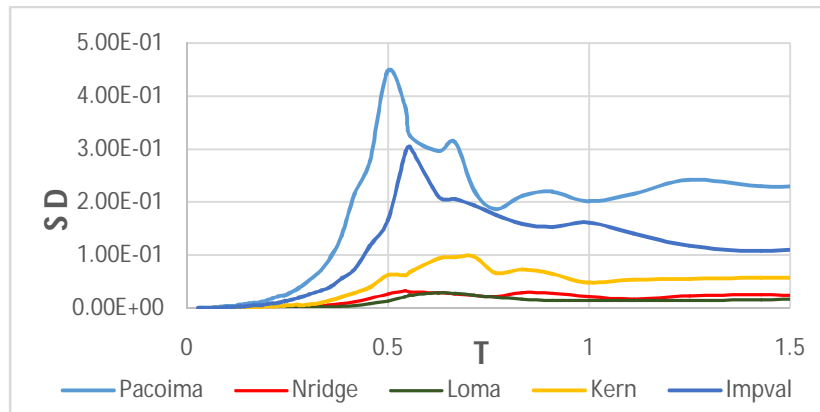


Fig 8. Displacement response spectra curve under 5% damping for structure at Darbhanga region

As per IS 1893(part 1): 2016 the maximum permissible storey drift in any storey shall not exceed 0.004 times the storey height i.e.  $0.004H = 36\text{mm}$ . The max displacement of structure located in Mangalore region along transvers direction due to earthquake load is 8.7mm and due to wind load is 30.74mm and along lateral direction due to earthquake load is 37.18mm. The max displacement of structure located in Darbhanga region along transvers direction due to earthquake load is 18.87mm and due to wind load is 35.7mm and along lateral direction due to earthquake load is 83.57mm. For both the location displacement due to wind load along lateral direction is negligible. The maximum displacement of structure located at both regions at equal time period is shown in fig 9.

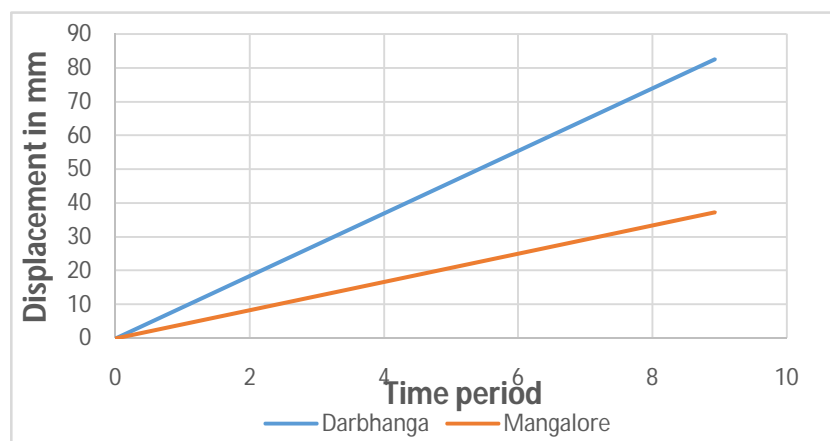


Fig 9. Maximum displacement of structure for both the regions

### IV. CONCLUSION

- Wind load combination is critical while designing the members of steel structures irrespective of highest wind zone or highest seismic zone depending upon location of the structure.
- The structure located up to wind zone III do not require any bracing whereas structure located in wind zone VI requires plan bracings.
- The structure located up to seismic zone III requires a simple bracings along lateral direction and clauses of IS codes will satisfy the design requirements whereas structure located in seismic zone V requires special type of bracing system and only IS code is not enough for the design the structure.

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