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Fragility curves for special truss moment frame with single and multiple vierendeel special segment

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Abstract

Special Truss Moment frame (STMF) is an open web truss moment frame, which dissipates the input seismic energy through a well-defined ductile special segment located near the mid-span of truss while other members of truss outside the special segment and columns are designed to remain elastic. In this paper, the performance and the fragility curve of STMFs consisting single and multiple vierendeel panels in the special segment are investigated. The seismic response of nine-story having the length to depth ratio of special segment 2.5 is considered to develop the fragility curve. The seismic response of each building was recorded by performing nonlinear incremental dynamic analyses. Each archetype modelled in nonlinear analysis program PERFORM-3D to carry out IDA under a suit of forty-four real Far Field ground motion records. Fragility curves were developed for these structures and the probability of exceedance at immediate occupancy (IO) level, Life safety (LS) level and Collapse performance (CP) level was assessed for two level of hazards, DBE level (10% probability of exceedance in 50 years) and MCE level (2% probability of exceedance in 50 years). For DBE level earthquake intensity, the probability of exceedance for the CP performance level of STMF building for both structure is marginal while at MCE level the probability of exceedance at CP performance level is 71% and 45% for single and multiple panels respectively.

Keywords: *Special truss moment frame, special segment, Incremental dynamic analysis, fragility curve.*

1. Introduction

Truss moment frames are relatively easy to construct as compared to conventional moment frames because of its simple detailing requirements for moment connections as compared to solid web girders. The mechanical and electrical ductwork may be installed through the open web spaces which provide a maximum height of the ceiling. These frames are lightweight and provide great lateral resistance which makes them appropriate for the longer spans for commercial and office building uses. In spite of these advantages, these frames were severely damaged during Mexico Earthquake (September 19, 1985). The observed damage included a number of localised failure in truss girders as well as columns which leads towards research on this type of frame for satisfactory performance during severe earthquake [1]. To improve seismic performance, the research on this type of framing system started during the nineties at the University of Michigan [2,3]. The seismic performance of truss moment frame is improved through energy dissipation by ductile segments. The ductile segment is the weak-zone of the truss girder which acts like a ductile fuse during seismic event, which is known as special segment. The special segment in the form of X-type [3] or a Vierendeel type [4] is located near the middle of the truss girder. X-type configuration of special segment (SS) consists web diagonal separated by web vertical and chord members while vierendeel type configuration of SS consists only chord members or intermediate vertical members with chord members. The seismic energy is being dissipated through the inelastic deformations within the SS only, while rest of the structure behaves elastically. In X-diagonal type configuration of SS, the energy is dissipated through axial yielding and buckling of diagonal members along with the formation of plastic hinges at the end of the chord members of the SS. In vierendeel type configuration of SS, energy is dissipated through the formation of plastic hinges at the end of the chord members in case of single vierendeel panel in SS while in case of multiple vierendeel panel in SS, the plastic hinges form at the end of the chord members of the SS as well as the intermediate vertical members within SS. Earlier during the nineties, the research carried out using built-up angle sections arranged back to back which was not sufficient during severe earthquake or mid-rise to high-rise buildings. To increase the capacity of SS, built-up channel section arranged back to back were tested at the University of Michigan [5]. Later, built-up channel sections were used to check the performance of the buildings [6] and the equation for expected shear strength is modified [7]. In recent years, to improve detailing of members within the special segment, various tests are conducted on the built-up double channel and hollow section along with full-scale truss girder [8,9]. Analytical studies are carried out by various researchers to understand its collapse resistance [10] and the effect of aspect ratio of SS [11].

In this paper, the study on the STMF building with single and multiple vierendeel panels in the SS is carried out from the probabilistic point of view. The seismic fragility of these structures is assessed in a probabilistic framework. For this, these two structures are designed and modelled in nonlinear analysis software PERFORM-3D. These modelled frames are performed for incremental dynamic analysis (IDA) under a suite of twenty-two pairs of ground motion records [12]. Fragility curves are derived using IDA results considering three performance levels viz. immediate occupancy (IO), life safety (LS) and collapse prevention (CP).

2. Review of design methodology

ANSI/AISC 341 [13] provides guidelines for the analysis and design of STMF buildings. The guidelines for the configuration of truss girder is summarized as (1) span length, and depth of truss girder shall be limited to 20 m (65 ft) and 1.8 m (6 ft) respectively; (2) the length of the special segment shall be 10% and 50% of truss span length with lower and upper limit respectively; (3) aspect ratio of any panel in SS shall not exceed 1.5 and not to be less than 0.67.

The design of the seismic-force-resisting starts with the calculation of base shear for that fundamental design time period is required. The approximate fundamental time period of the structure is calculated by Eq. (1)

$$T_a = 0.028h^{0.8} \tag{1}$$

where h is the height of the seismic-force-resisting system from base to the top level. The design base shear is calculated using Eq. (2) as:

$$V = C_s W \tag{2}$$

where C_s is the design seismic response coefficient which is calculated in accordance with ASCE/SEI-7 [14], and W is the effective seismic weight of the structure. The computed design base shear is distributed at each story as per the equivalent lateral force method. The design of this seismic-force-resisting system starts with the design of the member of the special segment. The members within the special segment, which include chord member and intermediate vertical member are designed for the forces which are obtained by performing the linear elastic analysis of the STMF building under the applicable load combinations.

Members of the truss girder outside the special segment and columns are designed to resist the combination of factored gravity loads, lateral loads including overstrength and applicable maximum expected shear strength of special segment. For this purpose, elastic segments from the study frames selected and linear elastic analysis is performed for the applicable load combinations. Expected vertical shear strength of special segment having single and multiple viendeel panels are computed from Eq. (3) and (4) respectively. The second term in the Eq. (4) reflects the presence of intermediate vertical member within SS.

$$V_{ne} = \frac{3.6R_y M_{nc}}{L_s} + 0.036EI_c \frac{L}{L_s^3} \tag{3}$$

$$V_{ne} = \left(\frac{3.6R_y M_{nc}}{L_s} + 0.036EI_c \frac{L}{L_s^3}\right) + \frac{m}{2}\left(\frac{3.6R_y M_{nv}}{L_s} + 0.036EI_v \frac{L}{L_s^3}\right) \tag{4}$$

where R_y is the yield stress modification factor; M_{nc} is the nominal flexural strength of chord members of the special segment; M_{nv} is the nominal flexural strength of the Intermediate vertical member within the special segment; E represents Young's modulus of elasticity of steel (2×10^5 MPa); m is the number of intermediate vertical members; I_c represents the moment of inertia of the chord member of the special segment *(mm⁴)*; I_v is the moment of inertia of the intermediate vertical members (mm⁴); L is the span length of the truss *(mm)*; L_s is the length of the special segment *(mm)*.

All the members of the study frame are designed as beam-column elements in accordance with ANSI/AISC-360 [15]. The members within the special segment, as well as columns, are designed to meet the compactness criteria.

3. Study frames: Design and modelling

In the present study, two nine-story STMF buildings are considered consisting one with single vierendeel panel and the other with two vierendeel (multiple vierendeel) panels within the special segment. The study buildings are the regular symmetrical buildings with 36.60 m by 36.60 m in the plan as shown in Fig. 1. The truss girders are placed on the perimeter of the building for each study building. The width of each bay in both directions is 9.15 m (30 ft.). The total height of the buildings is 39.66 m (130 ft.) having a height of each story is 4.27 m (14 ft.) except the first story which is 5.50 m (18 ft.) high.

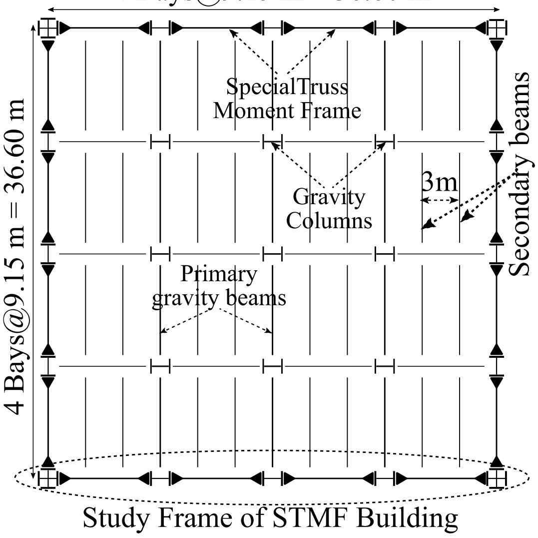

Fig. 1. Plan of the study buildings.

Fig. 2 shows the configuration of the truss girder for STMFs with single vierendeel panel in SS with dimensional details. The aspect ratio of SS is 2.5 and aspect ratio of vierendeel panel is also 2.5.

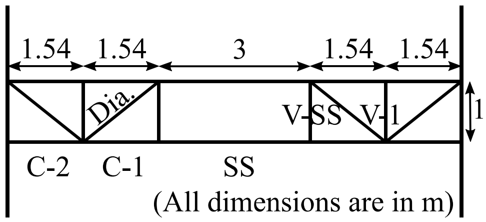

Fig. 2. Truss configuration of STMF with single vierendeel panel in SS.

Fig. 3 shows the configuration of the truss girder for STMFs with multiple vierendeel panels in SS by using the intermediate vertical members within the special segment. The aspect ratio of SS is 1.5 while the aspect ratio of vierendeel panel in special is 1.25. The depth of truss girder is 1.2 m for each configuration. Secondary beams are placed at one-third of span length of truss girder to avoid major structural load within the special segment.

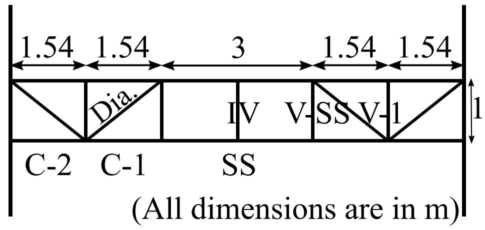

Fig. 3. Truss configuration of STMF with multiple vierendeel panels in SS.

The design of each archetype is carried out in accordance with the current provision of US code. The Dead loads on each floor of the building is assumed as 4.31 kN/m² from the slab and 1.29 kN/m² per unit height from perimeter curtain wall. Live loads on each floor of the building are assumed as 2.4 kN/m² except roof which is 0.96 kN/m². The seismic weight of each floor is computed as 6585 kN considering electrical appliances units and one penthouse on the roof. The seismic design parameter and site details are listed in Table 1. The design base shear coefficient is calculated on the basis of the approximate fundamental period using seismic parameters is computed as 0.062. The design base shear calculated from Eq. (2) is distributed at each floor using equivalent lateral force method. The member within the special segment is designed, by performing linear elastic analysis of the study frame under the applicable load combination of gravity loads and lateral loads. The member size of truss girder within the special segment are summarized in Table 2 for both buildings.

Table 1. Seismic Design parameters for buildings

Parameters	Value
Mapped MCE_R short period spectral acc. S_S	1.5g
Mapped MCE_R one-second spectral acc., S_1	0.6g
Site class	D
Short period site coefficient, F_a	1.0
Long period site coefficient, F_v	1.5
MCE_R spectral acc. at short periods, S_{MS}	1.5g
MCE_R spectral acc. at 1-sec period, S_{M1}	0.9g
Design spectral acc. at short periods, S_{DS}	1.0g
Design spectral acc. at 1-sec period, S_{D1}	0.6g
Occupancy Importance factor, I	1.0
Seismic design category	D
Response modification factor, R	7.0
Over-strength factor, Ω	3.0
Deflection amplification factor, C_d	5.5

The members outside the special segment are designed as non-yielding members. These non-yielding members are designed to resist the factored load combination of gravity loads, amplified lateral loads including overstrength factor and expected vertical shear strength acting at the mid-span of the truss girder. The members of the truss girders are designed assuming built-up channel section which is arranged back to back at a spacing of 25.4 mm (1 inch) as shown in Fig. 5(a). The chord members of the truss girder outside the special segment are designed assuming built-up channel section with extra web plates as shown in Fig. 5(b) wherever required. The buildings are assigned with an ID named as Bay width (ft.)_number of story_single vierendeel panel (S) and multiple vierendeel panels (M).

Table 2. Member size of truss girders within SS

Fl	30_9_S	30_9_M	
	SS	SS	V-I
9	C6X13	C6X13	C5X6.7
8	C7X14.75	C7X12.25	C6X8.2
7	C9X20	C8X13.75	C6X13
6	C10X20	C8X18.75	C6X13
5	C10X25	C10X20	C6X13
4	C10X25	C10X20	C6X13
3	C10X25	C10X20	C6X13
2	C10X30	C10X25	C6X13
1	C10X30	C10X25	C6X13

Note: FL: Floor, SS: Special Segment Chord, V-I: Intermediate vertical member

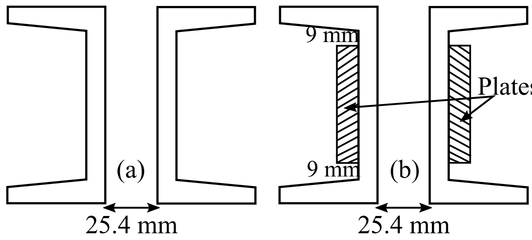

Fig. 4. Cross-section of members (a) within the SS, verticals, diagonals, chord (b) chord outside SS

The study frame indicated in Fig. 1 for each archetype is modelled as a two-dimensional frame in the nonlinear analysis software Perform-3D [16] with the aim of nonlinear modelling feature available in the software. The column bases of the study frame are assumed to be fixed. The gravity loads are transferred to the truss girder using secondary beams which act like point loads. ASTM A572 (Gr. 50) grade structural steel is assumed for all members of the study frame with a material yield stress of 345 MPa. Young's modulus of elasticity of structural steel and Poisson's ratio are assumed to be 200 GPa and 0.3 respectively. Lumped plasticity modelling is used for nonlinear modelling. Each member of the study frame is modelled as beam-column elements. The nonlinear behavior is incorporated in the elements through plastic hinges. The plastic hinges in each element are assigned at the end of the element, and between plastic hinges the element is assigned as elastic segment.

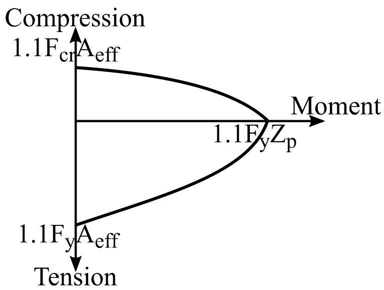

Fig. 5. Axial-moment interaction of members in special segments.

Fig. 5 shows the axial force (P) – bending moment (M_3) interaction surface of the elastic segment for each member and Fig. 6 shows the moment-plastic rotation response for the concentrated plastic hinges of the members within the special segment. The ultimate moment is taken as 1.4 times the yield moment considering yield stress modification factor. Force-deformation, as well as nonlinear features for the elements outside the special segment of truss girder, are modelled same as the SS chord member.

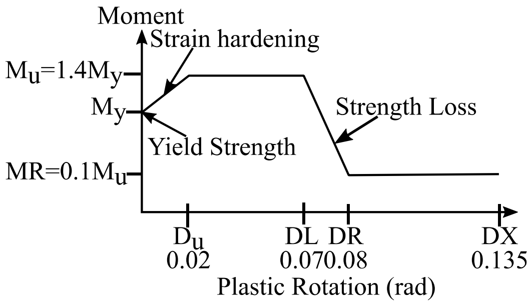

Fig. 6. Moment-plastic rotation curves for members in special segments.

The plastic hinges of the column elements are modelled in accordance with ASCE/SEI-41 [17] provisions assuming $P_u = 1.1P_y$ and $M_u = 1.1M_y$, where $P_y = R_yP_n$ and $M_y = R_yM_n$ (R_y = 1.1, P_n = nominal axial force capacity and M_n = nominal moment capacity). Rayleigh damping of 2% is assumed over a range of the time period of $0.2T_1$ to T_1, where T_1 is the fundamental time period of the structure.

4. Study frames: Analysis and Results

The nonlinear analyses are performed for each archetype with preloaded factored gravity load combination as:

$$1.05DL + 0.25LL \tag{5}$$

where DL represents the dead load and LL represents the live load imposed on the archetype. Nonlinear static analyses are performed to verify the behavior of the archetype and incremental dynamic analyses are performed for the development of fragility curve.

4.1. Nonlinear static analyses

The nonlinear static analysis (push-over analysis) is performed for each of the study frame under monotonically increasing lateral load with preloaded factored gravity loads combination of Eq. (5) until the control node of the roof exceeds the target displacement. Fig. 7 shows the pushover curves of study frame of both the buildings. Both the frame exhibits almost same lateral strength, stiffness and ductility. Both the frames are laterally pushed to the 5% of the roof drift. Plastic hinges first formed in intermediate vertical members and later progresses to the chord members of the special segment. It is observed that plastic hinges also formed in the vertical member near the chord member of the special segment. At 3.5% of roof drift the column starts yielding at sixth-story for 30_9_S while the fifth-story column starts yielding at 4.2% roof drift for 30_9_M.

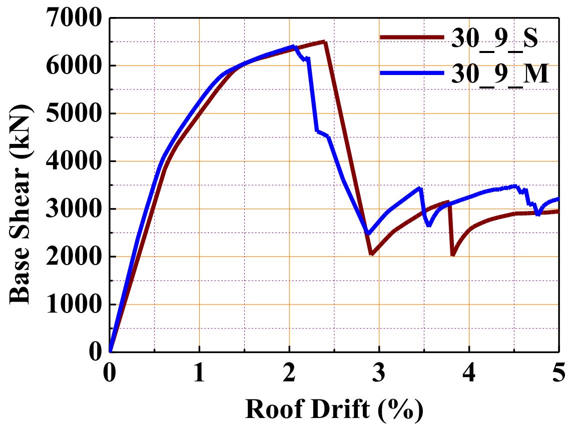

Fig. 7. Capacity curves for STMF buildings

4.2. Nonlinear dynamic analyses

The nonlinear time-history analysis is also conducted for each building to assess the response from linear elastic-phase to highly nonlinear state even at collapse under a suite of the gradually increasing ground motion intensity. To perform response history analyses, ground motion records of far-field (FF) record set of FEMA P695 [12] are considered. The FF ground motion record set includes twenty-two pairs of ground motion comprising in total forty-four individual ground motion. Fig. 8 shows the response spectra of FF record set including median response spectrum, the spectra representing one standard deviation and two standard deviation from median response spectrum.

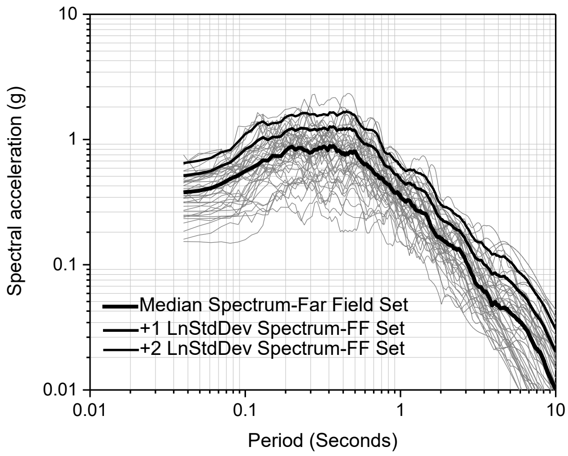

Fig. 8. Far-Field record set response spectra.

To develop fragility curves for the building results of the incremental dynamic analysis

(IDA) [18] are used. IDA was performed for each archetype under the factored load combination of Eq. (5) and scaled ground motions of FF record set. The scaled ground motions are used to cause the exceedance of damage from a threshold damage states. The scaling of ground motion records is carried out by anchoring the median spectral acceleration of the normalized records to a specific intensity such that median spectral acceleration matches with the spectral acceleration at the fundamental time period of the structure. The spectral acceleration varied from 0.05g to till the collapse of the structure (assuming 10% maximum IDR) [18] with an increment of 0.05g.

The maximum interstory drift ratio is recorded for a particular intensity of scaled spectral acceleration for each earthquake. These results are arranged in maximum interstory drift ratio versus scaled spectral acceleration for each earthquake creating the IDA curves as shown in Figs. 9 and 10. In Figs. 9 and 10, each curve represent the response of the building to a single ground motion record. As expected, STMF with multiple vierendeel panels in SS has limited the maximum IDR for low-intensity earthquakes.

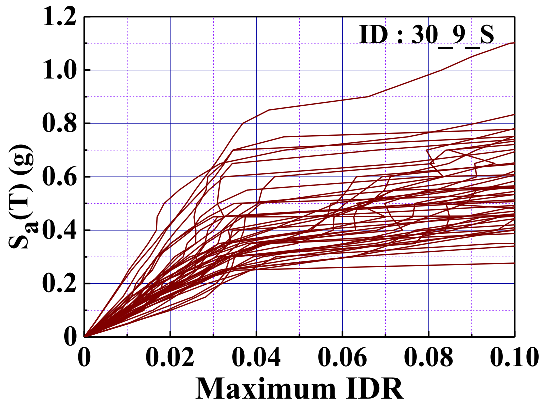

Fig. 9. IDA curves for 30_9_S building.

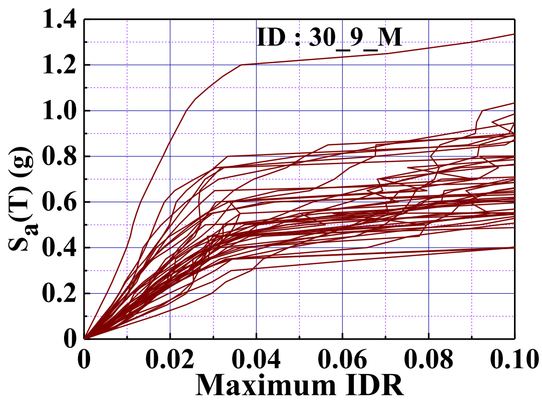

Fig. 10. IDA curves for 30_9_M building.

4.3. Development of fragility curves

The fragility curves in this section are developed for three performance levels namely immediate occupancy (IO), life safety (LS) and collapse prevention (CP). To evaluate the structural performance for these levels maximum interstory drift ratio is considered as the primary parameter as suggested by FEMA-356 [19]. In this study, the limiting transient interstory drift ratio was assumed same as for the steel moment frame corresponding to each performance level. FEMA-356 [19] suggests that the transient interstory drift limits for the IO, LS and CP are 0.7%, 2.5% and 5% respectively.

Kennedy et al. [20] defined that the seismic fragility of a structure is the conditional frequency of failure for a particular value of the seismic response parameter (e.g. stress, moment and spectral acceleration). The fragility function in this paper represents the cumulative distribution function (CDF) relating the intensity of spectral acceleration to probability of exceedance at a desired limit states. Assuming that the data obtained from IDA results are log-normally distributed, the fragility curves can be developed at desired limit states by using the median and logarithmic standard deviation from IDA results at predefined engineering demand parameter. The probability of exceedance can be analytically computed using Eq. (6) as defined in FEMA P695 [12]:

$$p\left[ds \middle| S_{aT}\right] = \Phi\left[\frac{1}{\beta_{RTR}} ln\left(\frac{S_{aT}}{S_{aT(50\%)}}\right)\right] \quad (6)$$

where $S_{aT(50\%)}$ represents the median value of spectral acceleration computed from IDA results at the desired limit of damage states, *ds*; β_{RTR} represents the standard deviation of natural logarithm of the spectral acceleration, S_{aT} for the damage state, *ds*, due to record-to-record variability; and Φ represents the standard normal cumulative distribution function. In this study, the spectral acceleration at the time period for the first mode of the structure ($S_a(T)$) and the maximum interstory drift ratio particular to three limits of damage states viz. Immediate occupancy (IO), Life safety(LS) and Collapse prevention(CP) are considered as S_{aT} and *ds* values for deriving the fragility curves.

The median spectral acceleration and lognormal standard deviation for each damage states are summarized in Tables 3 and 4. As expected that the median spectral acceleration at each performance level is higher for 30_9_M than 30_9_S

Table 3. Median spectral acceleration for buildings

Building ID	Median spectral acceleration (g) IO	LS	CP
30_9_S	0.068	0.249	0.391
30_9_M	0.09	0.36	0.52

Table 4. Lognormal standard deviation (β_{RTR}) for buildings

Building ID	Lognormal standard deviation (β_{RTR}) IO	LS	CP
30_9_S	0.34	0.34	0.30
30_9_M	0.38	0.35	0.26

The fragility curves of the buildings derived for three damage states IO, LS and CP are shown in the Figs. 11 and 12 considering only record-to-record variability.

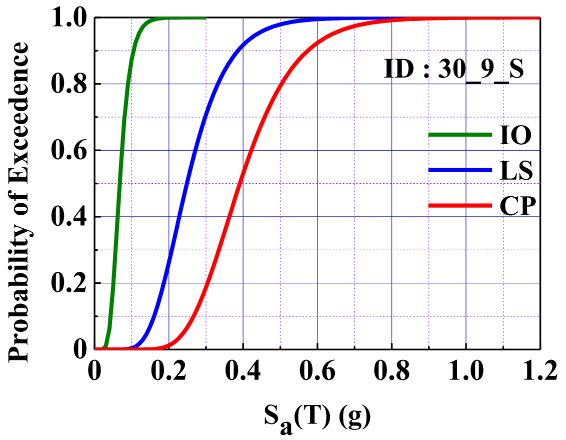

Fig. 11. Fragility curves of 30_9_S building.

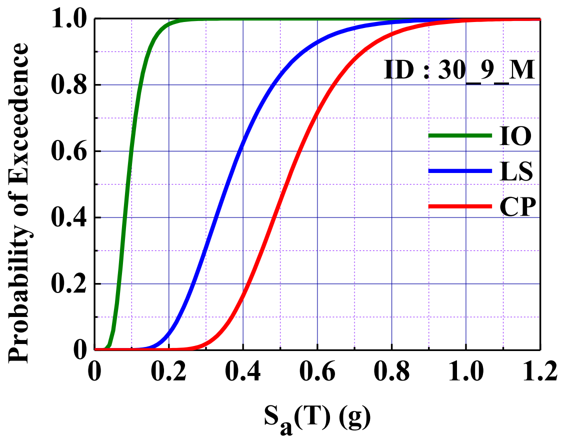

Fig. 12. Fragility curves of 30_9_M building.

4.4. Performance assessment

Based on the derived fragility curves, the performance of the buildings for three performance levels are evaluated considering two hazard levels viz. DBE and MCE level. Table 5 and 6 summarized the probability of exceeding performance level at DBE level and MCE level respectively. STMFs with multiple panels in SS perform better than the STMF with a single panel in SS at LS and CP. Both the frames show similar performance at IO performance level.

Table 5. Probability of exceeding performance level at DBE for buildings

Building ID	First mode period (T_1) in Second	DBE Spectral acc. at T_1 (g)	Probability of exceedance (%) at DBE Level IO	LS	CP
30_9_S	1.97	0.305	99	71	19
30_9_M	1.81	0.331	99	41	4

Table 6. Probability of exceeding performance level at MCE for buildings

Building ID	First mode period (T_1) in Second	MCE Spectral acc. at T_1 (g)	Probability of exceedance (%) at MCE Level IO	LS	CP
30_9_S	1.97	0.457	100	96	71
30_9_M	1.81	0.497	99	83	45

5. Conclusions

Special truss moment frame with single and multiple vierendeel panels in the special segment are performed through nonlinear static and incremental dynamic analysis. The pushover response of both buildings shows similar lateral strength while the progress of yielding is slightly different. In case of multiple vierendeel panels, yielding starts in intermediate vertical members and progress through the chord members of SS to columns. The start of yielding in column members are delayed due to the presence of intermediate vertical members.

The incremental dynamic analysis is performed to derive the fragility curves for each model at the immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance levels.

The probability of exceedance at two hazard levels is assessed at the performance levels. The two hazard levels with return period 475 (DBE level) years and 2475 years (MCE Level) are considered. The results of the fragility curve show that probability of exceedance at these two hazard levels is less at LS and CP performance limits for the STMF with the intermediate vertical member while at IO performance limits, both buildings perform similar.

It seems from the study that at DBE level the STMF building with multiple vierendeel panels in the special segment is safe while STMF with the single panel is not safe for collapse prevention. At MCE level both the buildings are not safe for collapse prevention.

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