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PROBABILISTIC ANALYSIS OF THE MR-FRAMES EQUIPPED WITH FREEDAM DAMPERS

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Abstract

The goal of this work is to provide a probabilistic approach to achieve a global collapse mechanism for a fixed goal reliability in moment-resisting frames equipped with "FREEDAM" dampers. First, a benchmark structure equipped with FREEDAM connections is designed through the Theory of Plastic Mechanism Control (TPMC). Then, the computation of the partial safety factors and the overstrength coefficient needed to account for the influence of random variability of column steel, friction coefficient and tightening torque in the seismic design of FREEDAM connections are reported. The probabilistic version of the Theory of Plastic Mechanism Control (P-TPMC) is applied to MR-frames equipped with FREEDAM connections to design the non-dissipative zones by considering aleatoric uncertainty on the material properties. The method proposed evaluates the probability of failure in the attainment of a collapse mechanism of global type. It constitutes the Probabilistic version of the Theory of Plastic Mechanism Control (P-TPMC). Stochastic frames have random values of the yield strength of members, so that the random variables are the plastic moments of both beam and column sections, thus randomly affecting the collapse mechanism developed under seismic horizontal forces. The selected study case has been performed by accounting for three different coefficients of variation equal to 0.05, 0.10 and 0.15. Results show that for increasing value of the overstrength factor R the probability of failure decrease.

Keywords: FREEDAM damper, MR-frames, collapse mechanisms, stochastic frames, probability of failure.

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1 INTRODUCTION

Moment Resting Frames (MRFs) equipped with FREEDAM dampers are one of the smartest solutions to design steel structures able to withstand frequent seismic events without significant damage and to remain safe, even though a certain amount of structural damage is accepted in case of rare seismic events [1]-[9].

In traditional structure the dissipation of energy provided by an earthquake is archived by the formation of plastic hinges developing wide and stable hysteresis loops in specific zone called dissipative zones. On the contrary other members, called non-dissipative zones, must remain in elastic range. In fact, on one hand, the structural damage is essential to dissipate the earthquake input energy, but for the other hand, is the main source of direct or indirect losses. Even if passive control strategic have been commonly based on the interpretation of the energy dissipation capacity on the primary structure, the main design strategies are devoted to the case of dissipative devices to substitute traditional dissipative zones. In case of MRFs, traditional dissipative zones are in beam ends, that can be easily substituted by friction dampers which can be located either at the level of the two flanges or at the bottom flange level only, being symmetrical or asymmetrical [10]-[14].

The aim of this work is to provide a probabilistic approach to achieve a global collapse for a fixed goal reliability in moment-resisting frames equipped with "FREEDAM" dampers so the Probabilistic version of the Theory of Plastic Mechanism Control (P-TPMC) is applied to MR-frames equipped with FREEDAM connections in order to design the non-dissipative zones by considering aleatoric uncertainty on the material properties, i.e, the yield strength of steel members, the static friction coefficient of the contact surfaces and the friction coefficient [15]-[21].

2 DESIGN OF FREEDAM JOINTS

FREEDAM joints are designed considering that the actions of frequent earthquakes no determine any slippage and plastic hinges do not develop in structural members. The resistance of the connection is calibrated from the maximum stress that the beam achieves under the maximum between the Ultimate Limit State (ULS) combination and the seismic condition concerning the Fully Operational LS (OP) Limit States [2], [7].

The design moment resistance corresponding to the slippage of the friction damper equipping the connection is given by:

$$M_{slip.Rd} = \frac{\mu_{s.lower} n_b n_s F_{p,lt} h_t}{\gamma_{Mf}} \tag{1}$$

where n_b is the number of bolts, n_s is the number of contact surfaces, $F_{p,lt}$ is the long term value of the bolt preloading due to the tightening, h_t is the lever arm, $\mu_{s.lower}$ is the 5% fractile of the static friction coefficient of the contact surfaces as affected by the coating process and γ_{Mf} is the partial safety factor.

In the case of FREEDAM dampers, whose friction pads are coated with M4 material [10], the values $\mu_{s.lower} = 0.69$ and $\gamma_{Mf} = 1.162$ can be used. For different coatings, specific experimental tests shall be carried out to establish the design value of the friction coefficient.

A local hierarchy criterion must be satisfied. This is the case of the beam which is located in series with the beam-to-column connection and the case of the non-dissipative components of the connection which are in series with the friction damper, i.e. the dissipative component of the connection. In these cases, the overstrength due to the random material variability could be properly considered by referring to the upper fractile (95% fractile) of the static friction coefficient. A more accurate evaluation can be developed by using a FORM analysis [15] where the limit state function is properly defined considering that the design goal of the local hierarchy criteria is, first of all, the prevention of the beam yielding and, in addition, the prevention of yielding in all the joint components made by plates and bolts. In other words, the slippage of the friction damper has to constitute the weakest joint component with a predefined probability level (success probability), i.e. reliability. In the case of friction pads coated with M4 material, the FORM analysis developed to this scope has pointed out, to assure reliability equal to 95%, the need to apply a central safety factor equal to 1.22 accounting for a coefficient of variation of the yield strength of steel equal to 0.10 and a coefficient of variation for the bolt tightening equal to 0.06 [22]. It must be considered that $\Omega_{Cd}$is the overstrength to be considered with respect to the design value rather than the mean value (as it is the case of the central safety factor), therefore $\Omega_{Cd} = 1.22 \times 1.28 = 1.56$. The central safety factor is defined as the ratio between the mean beams design values and the mean FREEDAM joints design values:

$$\gamma_0 = \frac{E[M_{b.R}]}{E[M_{f.R}]} \tag{2}$$

Therefore, the use of an overstrength factor $\Omega_{Cd} = 1.56$ in the application of capacity design criteria at local level assures that the probability of yielding of the beam end is no more than 5% [10].

In the case of FREEDAM connections, with friction pads coating made of M4 material, a safe side value equal to 1.28 can be suggested for design purposes [22]. It results from a mean value of the friction coefficient equal to 0.76, a 5% fractile of the friction coefficient equal to 0.69 and the adoption of a partial safety factor for defining the design friction resistance equal to 1.162 ($0.76 \times 1.162/0.69 = 1.28 \cong 1.30$).

$$M_{b.Rd} \geq \Omega_{Cd} M_{f.Rd} \tag{3}$$

where $M_{b.Rd}$ is the resistant moment of beam, $M_{f.Rd}$ is the FREEDAM device resistance.

By considering that FREEDAM devices have a haunch dimension, the section of the beam to be checked as to the moved and, consequently, Eq. (2) can be rewritten as follows:

$$M_{b.Rd} \geq \Omega_{Cd} M_{f.Rd} \frac{L-a}{L} \tag{4}$$

where L is the distance from the zero-moment point assumed equal to half beam length and a is the distance between the column and the verification section and it is the sum of the length of the haunch and half beam section.

3 PROBABILISTIC THEORY OF PLASTIC MECHANISM CONTROL

Structures can withstand several collapse mechanisms, but it is universally recognizing that the optimum seismic performances are be obtained when a collapse mechanism of global type occurs. The Theory of Plastic Mechanism Control (TPMC) is a code competitor design tool which exploits the kinematic theorem extended to second order effects by the so-called collapse mechanism equilibrium curve [23]. Even though a recent development of such design approach has been published [24], in this paper reference is made to the classical approach developed in the α-δ plane which has been several time applied to deterministic structures having different seismic resistant scheme and also to MRFs with FREEDAM connections.

TPMC can guarantee the development of the global type of mechanism, but even in this case undesired collapse mechanisms could occur when the effects of random material variability are taken into account. This is the case of stochastic frames, whose members have random plastic moments due to the random variability of the yield strength of steel.

The modern seismic codes to compensate the random material variability suggest the use of the overstrength factors: $\Omega_{rm} = 1.35$ and $\Omega_{sh} = 1.2$ for S275 steel grade.

A rigorous application of capacity design principles aimed to the control of the collapse mechanism requires, in case of stochastic structures, the combination of a rigorous deterministic approach with the procedures commonly suggested in structural reliability analysis. Therefore, this work aims at the development of an advanced method for the seismic design of stochastic MRFs.

The method proposed evaluates the probability of failure in the attainment of a collapse mechanism of global type. It constitutes the Probabilistic version of the Theory of Plastic Mechanism Control (P-TPMC) [15]-[21] already successfully developed for frames with deterministic material properties [25]-[27]. Stochastic frames have random values of the yield strength of members, so that the random variables are the plastic moments of both beam and column sections, thus randomly affecting the collapse mechanism developed under seismic horizontal forces.

The failure domain derives from all the possible collapse mechanisms leading to a manifold failure surface. It is important to underline that, within structural reliability analysis and within the contest of failure mode control, the term failure denotes the attainment of a collapse mechanism different from the global one. The requirements to be satisfied to prevent undesired collapse mechanisms are probabilistic events. At each event corresponds a limit state function representing a hyperplane in the space of the random variables, so that the failure domain is a manifold surface resulting from the intersection of the hyperplanes corresponding to the limit states of the single events. The correlation between the single limit state events results because some plastic hinges belong to many different mechanisms affecting the same structure.

The probability of failure is computed by means of Ditlevsen bounds [28] applying the theory of binary systems and considering that, from the structural reliability point of view, the limit states are events located in series.

With reference to the possible collapse mechanisms of MRFs under seismic horizontal forces, it is possible to observe that a kinematic mechanism can develop according to patterns of yielding corresponding to two mechanism typologies, namely upper partial and shear band [15] The total number of possible mechanisms, with the exclusion of the global one, is: $N_{tot} = \frac{n_s(n_s+1)}{2} + n_s - 1$ [15].

3.1 Failure events

Starting from TPMC in its traditional form, it is easy to rewrite the design conditions needed to prevent undesired collapse mechanism in a new form which, conversely, represents the occurrence of any failure event. The term failure is herein referred to the case in which the mechanism equilibrium curve corresponding to any undesired mechanism is located below that corresponding to the global mechanism. The failure events, thus identified, are random events constituting a series system of binary components. Each failure event can be expressed by means of the following safety margin parameter $E_{i_b.i_t}^{(t)}$ [15]:

$$E_{i_b.i_t}^{(t)} = \left(\alpha_{0.i_b.i_t}^{(t)} - \gamma_{i_b.i_t}^{(t)}\delta_u\right) - (\alpha_0^{(g)} - \gamma^g\delta_u) < 0 \qquad (5)$$

where all the possible mechanisms different from the global one has to be considered, so that in case of upper partial mechanisms (t=up), $i_t = n_s$ and $i_b = (2,3,\dots,n_s)$ while in case of shear band mechanism (t=sb), $i_t = (1,2,\dots,n_s)$ and i_b ranges from 1 to i_t.

The parameters $\alpha_0^{(g)}$ and $\alpha_{0.i_b.i_t}^{(t)}$ are the first order collapse mechanism multipliers of global mechanism and generic mechanism, respectively. Conversely, γ^g and $\gamma_{i_b.i_t}^{(t)}$ are the slopes of the mechanism equilibrium curves for global and generic mechanism, respectively, while δ_u is the design displacement compatible with the ductility supply of the structure. The safety margin is negative when failure occurs, i. e. an undesired mechanism develops. The number of the considered inequalities is equal to the whole number of collapse mechanisms different from the global one. Starting from the above consideration, to apply the First Order Reliability Method (FORM), some preliminarily assumptions are made:

1) plastic moments of members are jointly Gaussian random variables because of random variability of steel yield strength;

2) second order rigid-plastic analysis is carried out to include the influence of second order effects;

3) horizontal seismic forces have a triangular distribution increasing with height;

4) vertical loads are assumed as deterministic quantities;

5) the plastic moment of columns is independent of the axial load.

3.2 Design procedure

It has been pointed out how the probability of occurrence of a collapse mechanism different from the global one can be computed by considering as probabilistic events those undesired events related to the location, at the ultimate displacement, of the mechanism equilibrium curve of an undesired mechanism below the one corresponding to the global mechanism. It means that there are several “failure” events equal to the number of undesired mechanisms.

Therefore, to set up a design procedure assuring the collapse mechanism of global type with a given success probability, it is needed to combine the deterministic theory of plastic mechanism control (TPMC), with the probabilistic approach.

In case of MRFs, the column sections must be designed considering the design resistance of FREEDAM joints, including the effects of random material variability.

Therefore, TPMC can be properly combined with FORM and Ditlevsen bounds provided that the randomness in yield strength is accounted for a specific over-strength factor which has to be properly calibrated to lead to the desired probability in assuring the global type mechanism [23]. Due to the lack of space, the mathematical steps are not reported in this work, but they are presented in [15] with reference to the MR-Frames.

The design procedure can be suggested starting from the assumption that, neglecting strain-hardening effects, the maximum bending moment which the FREEDAM connections are able to transmit to the columns is given by $\gamma_{ov} M_{f.Rd}$ where $M_{f.Rd}$ is the resistance of FREEDAM joints and γ_{ov} is the overstrength coefficient to be calibrated according to the following steps:

a) Define the design goal as the probability level desired in assuring a collapse mechanism of global type. This is the reliability guaranteed in the development of the global mechanism;
b) Assume a preliminary attempt value of the over-strength factor, $\gamma_{ov} = 1.0$;
c) Design the structure according to the deterministic theory of plastic mechanism control;
d) Compute the probability of "failure" P_f by means FORM, this is the probability of failing in the attainment of the design goal. Therefore, the reliability in the development of the desired collapse mechanism i.e., the global mechanism, is equal to $R = 1 - P_f$;
e) If the reliability level obtained as result of step d) does not comply with the design goal defined in step a), with a given tolerance, increase the over-strength factor γ_{ov} and come back to step b). Conversely, if the reliability of the global type of mechanism is sufficient, the procedure can be stopped, and the column sections are assumed as those resulting from TPMC applied for last value of γ_{ov}.

4 CASE STUDY

The examined study case is referred to Moment Resisting Frames with 4-storeys. The bay span is equal to 6.00 m and the inter-storey height are equal to 3.50 m. The parametric data are:

- friction pad: M4 material [9];
- coefficient of variation of the *nut factor K* depending on the friction coefficient only $C_{v.K}$ equal to 0.06 or 0.10;
- coefficient of variation of steel, $C_{v.s}$ equal to 0.05, 0.10 and 0.15.

The span length is equal to 6.00 m, the inter-storey height is 3.50 m, the number of columns is 5 located strong axis. Beams are located with permanent load $G_k = 15.00\ kN/m$ and variable load $Q_k = 6.00\ kN/m$ that are used to deign the beams section considering the deflection check and to compute the normal stresses in the columns. The beams sections resulted are IPE 300. The horizontal seismic forces selected to have a triangular distribution starting from 10 kN to 40 kN. The rotation of column plastic hinges is taken equal to 0.04 rad. The steel grade considered in the analysis is S275. The analysis consists in four phases:

1. Design structure with traditional TPMC(δ) starting from the beams for vertical loads;
2. Evaluation of FREEDAM joint resistance (see Section 2);
3. Design structures with FREEDAM connections using the TPMC approach;
4. Probability evaluation.

4.1 Design example for 4-Storeys case

The scheme in the following figure shows the Moment Resisting Frame with four storeys, the loads in seismic combination $G_k + 0.30Q_k = 16.80\ kN/m$ and the horizontal forces with a triangular distribution. Starting from the beams designed to withstand vertical loads the preliminary columns sections have been calculated.

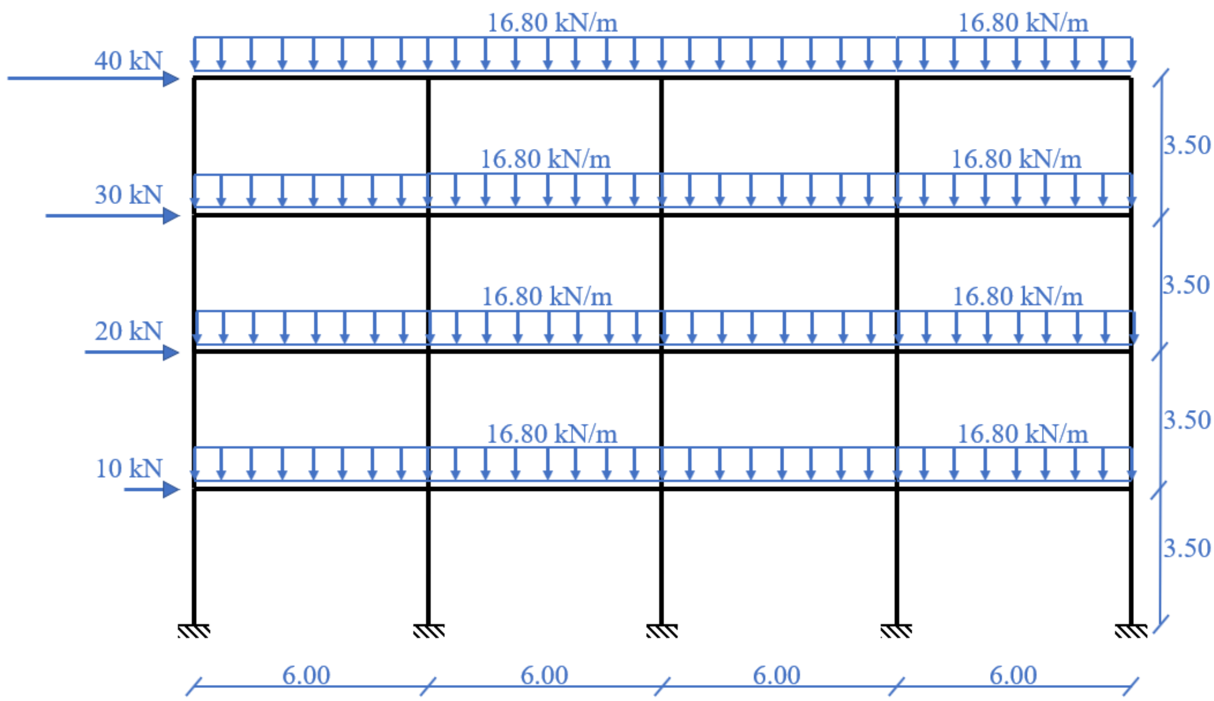

Figure 1: scheme of the 4 storeys Moment Resisting Frames

The moment developing at the beams end were obtained from the maximum between combination for vertical loads $(1.35G_k + 1.50Q_k)$ and the seismic combination that includes the

Fully Operational Spectrum calculated according to the new prEN1998 draft for the DC3 Ductility Class.

Storey	q_d [kN/m]	Beams	$M_{b.Rd}$ [kNm]	$\Omega_{rm}\Omega_{sh}M_{b.Rd}$ [kNm]	Columns	$M_{c.Rd}$ [kNm]
1	16.80	IPE 300	172.81	296.77	HE 340B	662.20
2	16.80	IPE 300	172.81	296.77	HE 340B	662.20
3	16.80	IPE 300	172.81	296.77	HE 340B	662.20
4	16.80	IPE 300	172.81	296.77	HE 300B	513.98

Table 1: structure designed without FREEDAM joints

The FREEDAM connections are calibrated directly by this maximum solicitation. For design choice the maximum between all the storeys is considered so the resistance of the joints is the same for all beams ends.

$$P_f = \frac{\gamma_{Mf} M_{f.Ed,max}}{\mu_{s.lower} n_b n_s h_t} \tag{6}$$

Device	L [mm]	H [mm]	B [mm]	x [mm]	$M_{f.Ed,max}$ [kNm]	P_f [-]	$M_{f.Rd}$ [kNm]
D1	505	260	221	170	84.29	38	84.84

Table 2: Design of FREEDAM joints

The local hierarchy criterion is checked according to Eq. (3). The resistance of FREEDAM joints is used in the TPMC procedure to calculate the columns sections of the final structure. In this case $\gamma_{ov} = 1$.

Storey	q_d [kN/m]	Beams	$M_{f.Rd}$ [kNm]	Columns	$M_{c.Rd}$ [kNm]
1	16.80	IPE 300	84.84	HE 220B	227.43
2	16.80	IPE 300	84.84	HE 220B	227.43
3	16.80	IPE 300	84.84	HE 220B	227.43
4	16.80	IPE 300	84.84	HE 200B	176.69

Table 3: structure designed with FREEDAM joints

For the application of TPMC the moment is calculated in static condition it occurs:

$$M_{f.Rd} = \gamma_{ov.ds} M_{fd.Rd} \tag{7}$$

By assuming that the variability of the static friction coefficient, the bolt preloading due to the tightening and the steel material can be expressed by means of gaussian variables, the research of the overstrength factor will pass through the central safety factor; the medium value of $M_{f.R}$ is given by:

$$E[M_{f.R}] = a\mu_s\mu_F \tag{8}$$

where μ_s and μ_F are, respectively, the mean value of the static friction coefficient and the bolt preloading due to the tightening, while a represent a costant value equal to the product of the number of the bolts, the sections and the heigh of level arm. The analyses have been performed amplifying. In this phase, only the resistant moment of the FREEDAM connections by means of the central safety factor γ_0 for columns design:

$$E[M_{c.E}] = TPMC\ (\gamma_0 E[M_{f.Rd}]) \tag{9}$$

Technological condition has not been applied to the design of the columns. So, failure probability would not respect a real project situation, but it gives just a relationship between central safety factor and TPMC application.

For a complete evaluation of the probability of failure are needed the vector that collects the moment of the devices, the vector of the standard deviation of the devices, the vector of the column resistance and the vector of the standard deviation of columns.

The vector collecting the moment of the devices: FREEDAM connections are exactly the same for each joint of the frame, therefore the elements of this vector have all the same value, equal to:

$$E[M_{f.R}] \tag{10}$$

Vector of the standard deviation of the devices Resistance of FREEDAM connections is the product of more than one Gaussian random variables and, in the general case, the incoming distribution from this operation is not a Gaussian random variable but it could be approximated to that. A problem is to search out a proper coefficient of variation to correctly describe the casual variable, with two point of the distribution, standard deviation is obtained as:

$$Dev[M_{f.R}] = a\sqrt{\mu_S^2\sigma_F^2 + \mu_F^2\sigma_S^2 + \sigma_S^2\sigma_F^2} \tag{12}$$

where σ_S and σ_F are, respectively, the standard deviation of the static friction coefficient and the bolt preloading due to the tightening the coefficient. Standard deviation of columns for each fixed $C_{v.s}$the standard deviation of each column is:

$$Dev[M_{c.R}] = E[M_{c.R}]C_{v.s} \tag{13}$$

In Table 4 and Figure 2 the results in function of the Reliability for the cov=0.05, 0.10 and 0.15 are reported.

γ_{ov}	**1.00**	**1.05**	**1.10**	**1.15**	**1.20**	**1.25**	**1.30**
Cov=0.05	0.5308	0.1360	0.0126	6.49E-04	3.95E-06	1.51E-08	2.94E-11
Cov=0.10	0.5445	0.2391	0.0883	8.16E-03	9.34E-04	7.92E-05	5.25E-06
Cov=0.15	0.6108	0.3784	0.2342	1.53E-01	9.46E-02	5.40E-02	2.50E-02

Table 4: Probability of failure

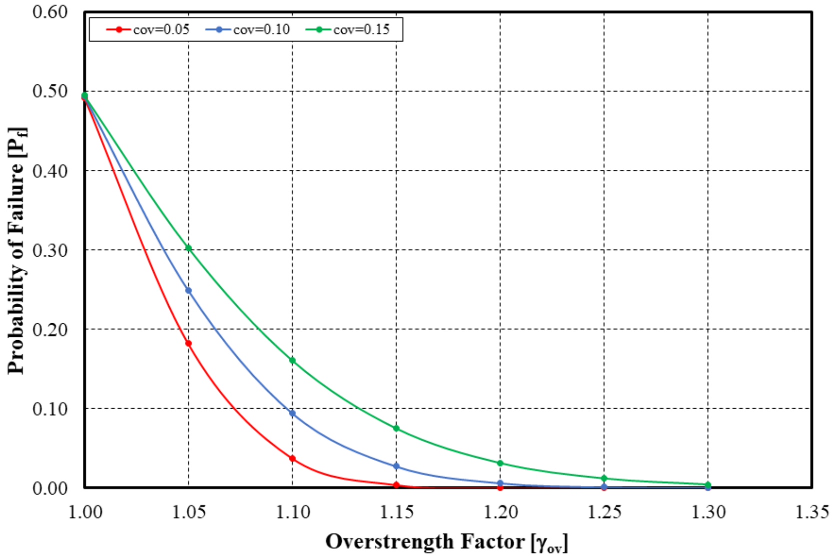

Figure 2: Trend of the probability as a function of "γ_{ov}" for cov=0.05, 0.10, 0.15

5 CONCLUSIONS

- In this work the Probabilistic Theory of Plastic Mechanism Control (P-TPMC), based on the combination of TPMC and FORM, was extended to Moment Resisting Frames equipped with FREEDAM connections.
- The combination of TPMC and FORM is based on an iterative procedure where the overstrength factor to be considered in the computation of the maximum bending moment that the FREEDAM joints can transmit to the columns is progressively increased up to the occurrence of the desired reliability level.
- Steel Moment Resisting Frame structure was designed first with traditional connections to evaluate the resistance of the FREEDAM joints and then with the friction devices by means of the TPMC.
- Steel Moment Resisting Frame structure has been performed by accounting for three different coefficients of variation equal to 0.05, 0.10 and 0.15. Results show that for increasing value of the overstrength factor R the probability of failure decrease.
- It is possible to notice that for cv=0.05 the probability of failure as a function of γ_{ov}=1.15 is of the 100%.
- Future study with different benchmark structures differing for the number of bays, storeys and measures will be developed.

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