Failure Mode Control and Displacement Based Design of Dissipative Truss Moment Frames: Seismic Performance Evaluation

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SUMMARY:

In this paper, a new approach to design Dissipative Truss Moment Frames (DTMFs) able to assure, under seismic forces, the development of a collapse mechanism of global type is applied and combined with Displacement Based Design procedures (DBD). The applied theory of plastic mechanism control, which is based on the kinematic theorem of plastic collapse, has been already successfully applied for other structural typologies and has been recently extended to the case of DTMFs. In particular, DTMFs constitute a particular case of Special Truss Moment Frames (STMFs), where the energy dissipation is provided by means of special dissipative devices located at the ends of truss girders at the bottom chord level. The effectiveness of the proposed design approach for failure mode control has been already investigated by means of nonlinear static and dynamic analyses, which have demonstrated the attainment of the design goal, i.e. the development of a collapse mechanism of global type involving all the dissipative devices. In this paper, this approach is combined with the application of Capacity Spectrum Method (CSM) aiming to the calibration of the stroke of the special dissipative devices located at the ends of truss girders.

Keywords: Dissipative Truss Moment Frames, kinematic theorem of plastic collapse, dissipative devices.

1. INTRODUCTION

STMFs constitute a quite recent development of traditional MRFs where the beams are replaced with truss girders having dissipative zones constituted by special segments located in the mid-span of the truss girder (Goel and Itani, 1994; Basha and Goel, 1995; Chao et al., 2008). Conversely, starting from traditional Truss Moment Frames (TMFs), DTMFs are simply obtained by inserting friction or hysteretic devices at the bottom chord level at the ends of each truss girder. Therefore, two specifically designed dissipative zones are introduced for each truss girder by properly exploiting the benefits coming from the use of energy dissipation devices which can be easily substituted after the occurrence of destructive earthquakes and, in addition, are able to prevent damage to the primary structural system (Kelly et al., 1972; Skinner et al., 1975; Filiatrault and Cherry, 1990). Therefore, in case of DTMFs, truss girders and columns constitute the non-dissipative zones and must be designed in order to remain in elastic range even in the case of destructive earthquakes, while the special devices constitute the dissipative zones where the seismic energy dissipation has to occur. The best seismic behaviour for the considered structural typology is obtained when all the dissipative devices located at the truss girder ends are “yielded”, i.e. involved in the kinematic mechanism. To this scope, a design methodology devoted to the failure mode control of DTMFs has been developed aiming to obtain a structure able to involve, in the collapse mechanism, all the local ductility sources represented by the dissipative devices and to avoid yielding of the primary structural elements. The proposed design methodology is the extension to DTMFs of a rigorous methodology already developed for failure mode control of Moment Resisting Frames (Mazzolani and Piluso, 1997) and successively extended to eccentrically braced frames (EBFs) (Mastrandrea and Piluso, 2009) and to Knee Braced Frames

(KBFs) (Conti et al., 2009). The aim of the proposed design methodology is the development of a global collapse mechanism assuring the participation of all the dissipative devices to the dissipation of the earthquake input energy. All the columns remain in the elastic range with the only exception of base sections of first storey columns where plastic hinges are needed for the complete development of a kinematic mechanism. In fact, even though dissipative devices are located at the ends of each truss girder, common hierarchy criteria do not assure that all of them are involved in the energy dissipation process, due to the development of partial mechanisms which engage the devices of only a limited number of storeys. The combination of a rigorous design methodology, based on the kinematic theorem of plastic collapse with the use of dissipative devices allows to assure that dissipation of seismic input energy occurs only in friction or hysteretic devices without the involvement of the main structure. In other words, the best seismic performance of DTMFs is reached when a global collapse mechanism is achieved (Fig. 1), whose development is the primary goal of plastic design of seismic–resistant structures.

The proposed design method has been presented in previous works (Longo et al. 2009, 2011) providing all the details for its practical application (Longo et al., 2012). In addition, a preliminary validation of the design procedure has been carried out, with reference to several designed structures, by means of non-linear static and dynamic analyses (Longo et al. 2009, 2011, 2012). In this paper, particular attention is devoted to the calibration of the stroke of devices. This design issue can be faced by means of Capacity Spectrum Method (Fajfar, 1999), comparing the capacity curve of the structure obtained by means of a Push Over Analysis, with the demand curves derived from the elastic design spectra, in ADSR format, provided by the seismic code. In particular, by means of the relationship between the sway displacements of the actual structure and the sway displacement of its equivalent SDOF system, it is possible to estimate the displacement demand required to satisfy a given limit state for a given seismic intensity measure, selected according to the seismic hazard of the site. The displacement demand of the equivalent SDOF system is related to the displacement demand of the actual MDOF structure and, as a consequence, to the maximum displacement demand of the dissipative devices whose stroke is design accordingly.

In addition, the results of incremental non linear dynamic analyses carried out with reference to the examined structure using OpenSees computer program (1999), have been compared with the displacement demands predicted by the application of CSM which is the tool to design the stroke of the dissipative devices located at the ends of truss girders at the bottom chord level in the proposed structural typology.

2. DESIGN METHODOLOGY

The proposed design procedure is based on the kinematic theorem of plastic collapse and on second order plastic analysis. It starts from the observation that collapse mechanisms of the considered structural typology subjected to horizontal forces can be considered as belonging to three main typologies, where the collapse mechanism of global type is a particular case of type-2 mechanism (Longo et al. 2012). The control of the failure mode can be performed by means of the analysis of $3n_s$ mechanisms (being n_s the number of storeys). The method starts from the knowledge of truss girder sections and of the resistance of the dissipative devices. The truss girders are designed to resist vertical loads, while the threshold resistance of the dissipative devices is chosen to be less than the axial resistance of the chords assuring the prevention of yielding or buckling in the structural elements of the truss girder. The unknowns of the design problem are the column sections whose plastic modulus has to be defined so that the kinematically admissible multiplier of horizontal forces corresponding to the global mechanism has to be less than those corresponding to the other $3n_s$-1 kinematically admissible mechanisms. According to the upper bound theorem, the above stated multiplier is the true collapse multiplier, so that the global failure mode is the mechanism actually developed. In particular, it is imposed that the mechanism equilibrium curve (α–δ) corresponding to the global mechanism has to lie below the equilibrium curves corresponding to all the other undesired mechanisms within a displacement range compatible with the local ductility supply of dissipative elements. This approach allows the prevention of column yielding, taking into account also second order effects (Mazzolani and Piluso, 1997; Longo et al., 2012).

3. APPLICATION

In order to evaluate the accuracy of the proposed design methodology, an adequate number of DTMFs having different numbers of storeys (4÷10) has been designed. In particular, for sake of shortness, only the results of an eight-storey DTMF will be discussed in this paper.

The building plan configuration is symmetric with reference to the two orthogonal directions (Fig.3.1), as a consequence, neglecting the accidental torsion due to the random variability of live load location, the distribution of the seismic horizontal forces among the seismic resistant systems is immediately obtained. S275 steel grade has been adopted. For each floor the dead load (G_k) is equal to 3 kN/m^2 and the live load (Q_k) is equal to 2 kN/m^2.

In Table 3.1 the member sections of truss girders and the column sections resulting from the application of the proposed design procedure are given. Regarding the elements of the truss girder, the spacing between UPN profiles (i.e. the thickness of the gusset plate) is equal to 15 mm. In the same table, the buckling resistance $N_{b.Rd}$ of chords and diagonals are also pointed out. For sake of synthesis only the case of an eight storey frame with threshold resistance of dissipative devices equal to 665 kN (corresponding to 50% of the buckling axial resistance $N_{b.Rd}$ of the chords of the truss girders) is herein presented, although the analyses have been carried out with reference to DTMFs with different values of the threshold resistance of dissipative devices.

Table 3.1 Results for the 8th storey DTMF with threshold resistance of dissipative devices equal to 665kN

Storey	Chords $N_{b.Rd}$=1330kN	Diagonals $N_{b.Rd}$=788kN	External columns [mm]	Internal columns [mm]
1	2UNP 240	2UNP 140	SHS 700x22	SHS 780x26
2	2UNP 240	2UNP 140	SHS 640x20	SHS 720x24
3	2UNP 240	2UNP 140	SHS 580x20	SHS 660x22
4	2UNP 240	2UNP 140	SHS 580x20	SHS 660x22
5	2UNP 240	2UNP 140	SHS 580x20	SHS 660x22
6	2UNP 240	2UNP 140	SHS 560x19	SHS 640x20
7	2UNP 240	2UNP 140	SHS 500x18	SHS 560x20
8	2UNP 240	2UNP 140	SHS 400x14	SHS 460x16

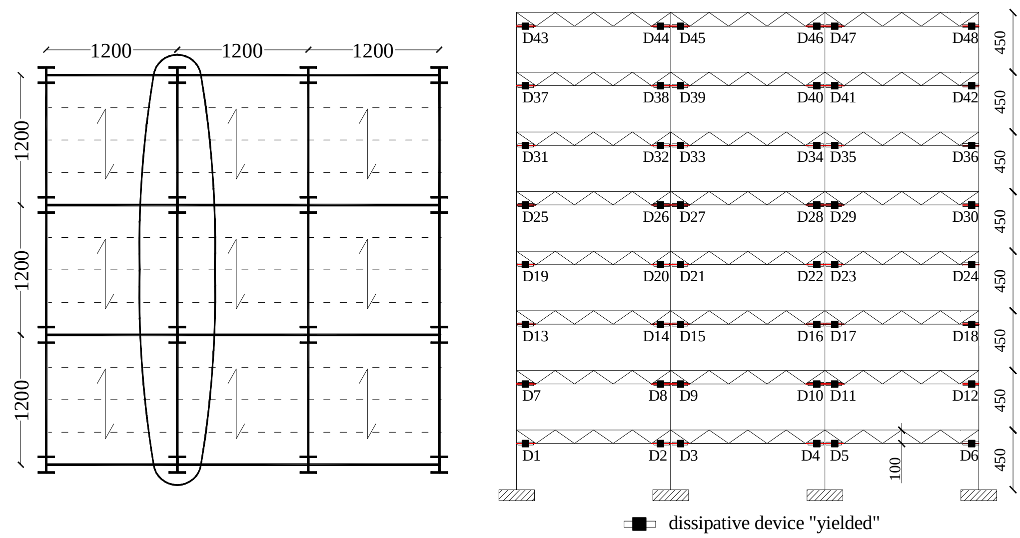


Figure 3.1. Analysed structure and numbering of dissipative devices

4. CAPACITY SPECTRUM METHOD

After severe earthquakes, especially after Northridge and Kobe earthquakes, researchers have been spurred towards the improvement of design rules provided by seismic codes. In U.S.A., the structural engineering community has been involved in the process of developing a new generation of design and rehabilitation procedures incorporating performance-based engineering concepts (UBC, 1997; NEHRP, 2001a; NEHRP, 2001b). Among them, the popularity of the so-called Capacity Spectrum Method (CSM) has increased quickly.

By means of graphical procedures, the CSM compares the structural displacement capacity with the displacement demand due to the earthquake ground motion. The capacity of the structure is represented by means of a force-displacement curve, obtained by a non-linear static (push-over) analysis. Base shear forces and top sway displacements are converted into spectral accelerations and spectral displacements, respectively, of the elastic-plastic equivalent SDOF system. These spectral values define the capacity spectrum. The demands due to the earthquake ground motion are defined, for each considered limit state and for the defined seismic zone, in terms of elastic spectra. The Acceleration-Displacement Response Spectrum (ADRS) format is used, providing spectral accelerations versus spectral displacements. The intersection between the capacity spectrum and the demand spectrum provides an estimate of the inelastic acceleration and displacement demand.

In such a way it is possible to determine the maximum displacement demand of the equivalent SDOF system and, by means of the relationships connecting the response of MDOF structures to the equivalent SDOF system, the maximum design displacement of the actual MDOF system can be obtained.

Two seismic force distributions are usually considered in performing the push-over analysis of the structure. The first force distribution is proportional to the storey masses (I distribution) whereas the second one (II distribution) is proportional to the product between the storey masses and the storey displacement corresponding to the first vibration mode of the structure.

For each force distribution, the method can be applied by means of the following steps:

1) definition of the capacity curve in terms of top sway displacement (d) and base shear forces (V_b) by means of a push over analysis of the MDOF system;
2) definition of the capacity curve of equivalent SDOF system by means of the following relation:

$$F^* = \frac{V_b}{\Gamma}\,;\; d^* = \frac{d}{\Gamma} \tag{4.1}$$

where $\Gamma = \sum_{i=1}^{N} m_i \cdot \phi_i / \sum_{i=1}^{N} m_i \cdot \phi_i^2$ is the participation factor of first vibration mode and V_b is the value of the base shear force. Successively, the capacity curve of the equivalent SDOF system can be approximated with a bilinear curve by means of the following relation:

$$d_y^* = \frac{F_y^*}{k^*} \tag{4.2}$$

where F_y^* and d_y^* are the coordinates of the yield point of the bilinear curve system and k^* is the stiffness of the elastic branch. k^* can be evaluated by imposing the intersection between the capacity curve of the equivalent SDOF system and the elastic branch of the bilinear curve at the value of $0.60V_{bu}$, being V_{bu} the maximum value of the F^*-d^* curve. F_y^* can be evaluated by imposing the equality of the area under the F^*-d^* curve and the bilinear curve up to a value included in the range (0.85-1.00) V_{bu} on the softening branch of F^*-d^* curve.

The vibration period of the equivalent SDOF system (T*) is equal to:

$$T^* = 2\pi\sqrt{\frac{\sum_{i=1}^{n} m_i \phi_i}{k^*}} \tag{4.3}$$

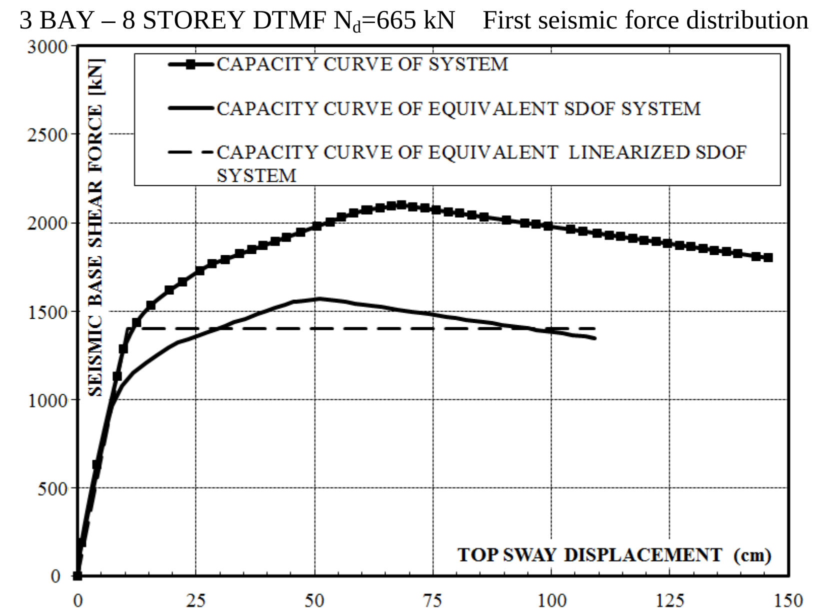
3) Using the elastic spectrum in terms of displacements, the maximum displacement response of the bilinear equivalent SDOF system can be determined. If the vibration period of the equivalent bilinear SDOF system T* is greater than T_c (where T_c is the limit value of the constant spectral acceleration branch) the displacement demand of the inelastic system is assumed equal to an elastic equivalent system having the same period ($d^* = d^*_{e.\max} = S_{De}(T^*)$). In the opposite case, if T*< T_c, the displacement demand of the inelastic system is greater than the one corresponding to

the elastic system. In this case the displacement demand is evaluated by means of following relation:

$$d^*_{\max} = \frac{d^*_{e.\max}}{q^*}\left[1+(q^*-1)\frac{T_c}{T^*}\right] > d^*_{e.\max} \tag{4.4}$$

where $q^* = \frac{S_e(T^*)}{F_y} m^*$ and $m^* = \sum_{i=1}^{N} m_i \cdot \phi_i$ is the mass of the equivalent SDOF system.

In this paper, the ADSR spectrum provided by Italian Seismic Code (D.M. 14/01/2008) with reference to the seismic zone of Reggio Calabria - South Italy, corresponding to a seismic intensity having 5% probability of exceedance in 100 years (1950 years return period) has been considered. This seismic intensity is referred to the collapse prevention (CP) limit state and to a building reference life equal to 100 years. In addition, a soil type A (stiff soil conditions) and topographic coefficient equal to 1.4 have been used.



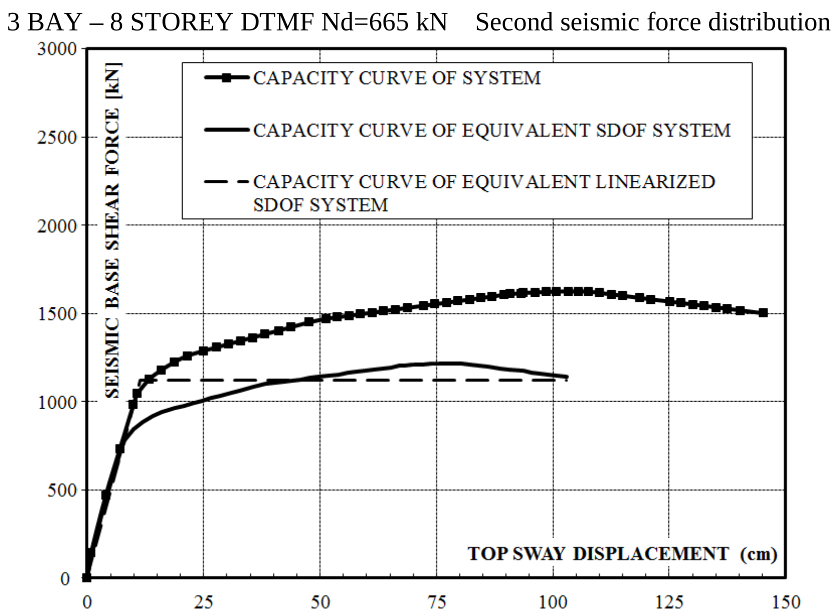


Figure 4.2. Push-over curves and bilinear approximation

With reference to the designed eight storey DTMF (Table 3.1) the push-over analyses carried out using the two seismic force distributions provided by the code (the first one is proportional to the storey masses whereas the second one is proportional to the product between the storey masses and the deformation shape of the structure corresponding to the first vibration mode) have been performed.
The push-over analyses have been carried out by means of SAP2000 computer program (2007). In particular, columns have been modelled using beam-column elements with the possibility of developing plastic hinges at their ends. Moreover, the truss girders have been modelled using truss elements having the possibility of yielding under axial forces. Finally, also the dissipative devices have been modelled by means of non-linear truss elements whose "yield" axial load represents the axial threshold resistance of the device.
The push-over curves and the corresponding bilinear approximations obtained with reference to the eight storey DTMF are depicted in Fig. 4.2. Starting from these curves, by means of Eq. (4.1) and (4.2), the push-over curves of the equivalent SDOF system and the corresponding bilinear approximations are obtained for each seismic force distribution.
In Fig. 4.3, the ADSR spectrum corresponding to the seismic intensity measure for the collapse prevention limit state (CP) and to the selected Italian seismic region is overlapped to the bilinear approximation of the capacity curves of the equivalent SDOF system for the two horizontal seismic force distributions. The intersection points between the extension of the elastic branches of the equivalent SDOF systems and the ADSR spectrum provide the displacement demands of the equivalent SDOF systems *(d*)* for the given seismic zone. By means of Eq. (4.1) the displacement demand of the analysed eight DTMF structure (*d*) can be evaluated.

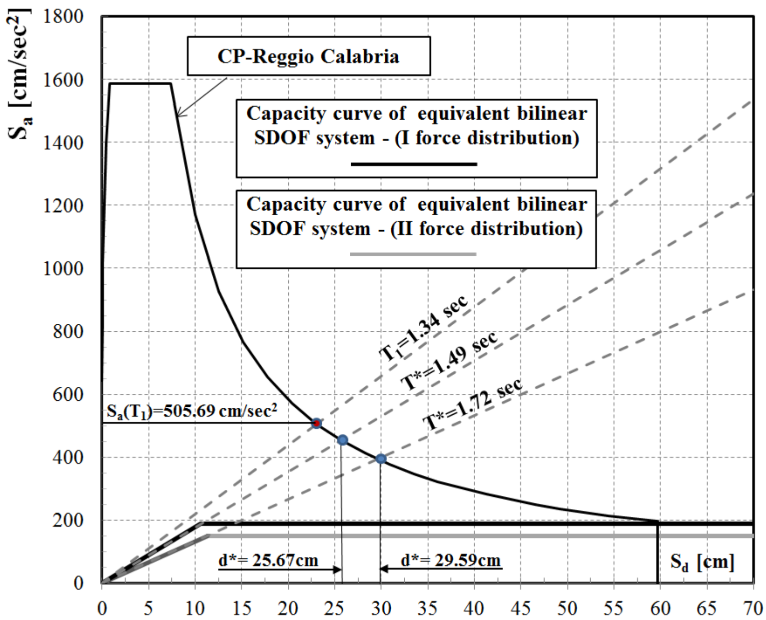


Figure 4.3. Application of CSM to the designed DTMF for Reggio Calabria seismic zones

Table 4.2 provides the results obtained for the two considered seismic force distributions in terms of displacement demand of the equivalent SDOF system *(d*)*, displacement demand *(d)* of the actual MDOF system (eight storey DTMF) and corresponding drift θ (i.e. the ratio between the displacement demand of the structure *d* and the total height *H* of the structure). In addition, in the same table all the parameters defining the equivalent bilinear SDOF systems have been reported.
It is possible to note that the second force distribution provide, for the considered spectrum, the highest value of the displacement demand and, as a consequence, the most severe value of the required drift ratio. In particular, the required drift ratio is equal to 0.95% for first seismic force distribution and 1.10% for the second seismic force distribution. This means that, with reference to the collapse prevention limit state, the value of the required stroke of the dissipative devices located at the ends of the truss girders is equal to ±0.011 x 1000 = 11.00 mm for Reggio Calabria seismic zone, being the lever arm of the truss girders, i.e. the distance between the chords, equal to 1000 mm.

Table 4.2 Results of capacity spectrum method for the designed DTMF

<table>
<tr><td colspan="6">Reggio Calabria Seismic Zone (T_r=1950 years)</td></tr>
<tr><td colspan="3">I seismic force distribution</td><td colspan="3">II seismic force distribution</td></tr>
<tr><td colspan="3">Γ=1.336</td><td colspan="3">Γ=1.336</td></tr>
<tr><td colspan="3">F_y*=1399.55 kN</td><td colspan="3">F_y*=1122.70 kN</td></tr>
<tr><td colspan="3">d_y*=10.60cm</td><td colspan="3">d_y*=11.29 cm</td></tr>
<tr><td colspan="3">K*=132.03 kNcm</td><td colspan="3">K*=99.44 kNcm</td></tr>
<tr><td colspan="3">m*=7.47 kN sec^2/cm</td><td colspan="3">m*=7.47 kN sec^2/cm</td></tr>
<tr><td colspan="3">T*=1.49 sec</td><td colspan="3">T*=1.72 sec</td></tr>
<tr><td>d* [cm]</td><td>d [cm]</td><td>θ [rad]</td><td>d* [cm]</td><td>d [cm]</td><td>θ [rad]</td></tr>
<tr><td>25.67</td><td>34.30</td><td>0.95%</td><td>29.59</td><td>39.53</td><td>1.10%</td></tr>
</table>

5. INCREMENTAL NON LINEAR DYNAMIC ANALYSES

In order to check the accuracy of the proposed design structure, the seismic response of the structure has been investigated by means of nonlinear dynamic analyses carried out using OpenSees computer program (1999) which allows to model the structural elements using nonlinear fibres elements.
The dissipative devices are modelled using zero length spring elements having the possibility to reproduce the hysteretic behaviour of the dissipative devices by means of an appropriate calibration of the “yield” threshold. Out-of-plane stability checks of compressed members have been carried out for each step of the analysis according to Eurocode 3 (2005).
Aiming to perform incremental dynamic non-linear analyses (IDA) all the records have been properly scaled to provide increasing values of the spectral acceleration $S_a(T_1)$ corresponding to the fundamental period of vibration of the structure, equal to T_1*=1.34 sec*. In particular, the analyses have been repeated increasing the $S_a(T_1)$ value until the occurrence of structural collapse, corresponding to column, chord or diagonal buckling, or to the complete development of a collapse mechanism or up to the attainment of the limit value of the peak interstorey drift ratio (PIDR) assumed equal to 0.04 rad. This last value is also the maximum plastic interstorey drift angle assumed for the evaluation of the design displacement to be used in the design algorithm for failure mode control (Longo et al., 2012).
In Table 5.3 the ground motions used for Incremental Dynamic Analyses are reported. It is important to underline that the spectra of the considered ground motions are compatible with the one provided by Italian Seismic Code.
By means of IDA the actual behaviour of the structure can be investigated. In particular, in Fig.5.4, the maximum interstorey drift ratio MIDR (i.e. the maximum PIDR among the different storeys) versus spectral acceleration has been reported. In addition, in the same figure, the value of the spectral acceleration $S_a(T_1)$ corresponding to the period of vibration of the structure (T_1 *=1.34 sec*) and to 5% probability of exceedance in 100 years, for the considered Reggio Calabria site, has been pointed out. For this selected value of the spectral acceleration the MIDR values exhibited by the actual MDOF system can be evaluated for each considered ground motion. In particular, MIDR values equal to 0.80%, 1.25%, 1.50% and 1.70% are obtained for Friuli, Gazli, Tokyo and Helena ground motion records, respectively. As expected, these results point out a significant scatter due record-to-record variability of the seismic input.
This variability source is not accounted in the codified version of the capacity spectrum method, where the demand values provided by CSM have to be considered just as a rough estimate of the mean value of the seismic demand.
Therefore, in order to compare the results provided by CSM and IDA, the average value of MIDR provided by the four considered ground motions has been considered. This mean value is equal to 1.31%, so that it can be concluded that in the examined case the percentage difference between CSM results and IDA results is equal to -16%. It is an acceptable accuracy compared with the simplicity of the capacity spectrum method which can be suggested as a simple approach to design the stroke of the dissipative devices. However, it has also to be underlined that this accuracy is probably due to the benefits coming from a structure designed to assure a collapse mechanism of global type, i.e.

structures which, more than structures designed with other criteria, can be modelled with an equivalent SDOF system.

Table 5.3 Set of historical ground motions used for IDA

Record	Date	Component	Length [sec]	a_{max}/g	$S_a(T_1)/g$
Gazli USSR (Karakyr)	17/05/1976	N-S	16.25	0.608	0.4391
Helena MONTANA (Carrol College)	31/10/1935	E-W	9.67	0.153	0.1467
FRIULI (San Rocco)	15/09/1976	N-S	16.92	0.035	0.0889
TOKYO	1956	N-S	11.40	0.075	0.0364

In addition, in this paper, also the effectiveness of the design methodology has been investigated by means of the interpretation of the results deriving from IDA.
The main goal of the analyses performed is to gain information on the elements involved in the seismic energy dissipation. In particular, before the development of the collapse mechanism, all the dissipative devices are involved in the seismic energy dissipation whereas all the columns and all the truss girders are in elastic range. For each accelerogram, the MIDR reached the limit value equal to 0.04 rad before the complete development of a collapse mechanism. Finally, for each dissipative device, in Table 5.4 the seismic energy dissipated and the corresponding cumulated plastic excursion (given by the ratio between the total energy dissipated by the device and its yield threshold) have been reported with reference to Helena accelerogram scaled at $S_a(T_1)=1.4\ g$ corresponding to a PGA equal to 1.46 g leading to the limit value of the drift angle. In the same table, the seismic structural demand expressed in terms of maximum displacement required to devices (stroke) are also pointed out. In particular, for Helena record scaled as specified above, the maximum required displacements are equal to 40.18 mm testifying that the stroke of the dissipative devices can be easily obtained by an appropriate design of such devices. In particular, it is important to underline that such values are in excellent agreement with the product between the design plastic rotation of base columns, equal to 0.04 rad, and the lever arm of truss girders, equal to 1000 mm. Furthermore, all the devices are involved in the seismic energy dissipation, because the cumulated plastic excursion of the devices is always greater than zero whereas all the primary structural elements are in the elastic range. Therefore, it can be concluded that the design goal has been actually obtained.
The above briefly summarised results have been obtained for all the analysed ground motion records confirming the accuracy of the proposed design methodology.
This result represents an excellent seismic performance, because when the limit value (0.04 rad) of the interstorey drift is achieved the PGA is very large and the collapse mechanism is not yet developed.
Moreover, after destructive earthquakes, dissipative devices can be substituted, provided that permanent deformations can be recovered by bringing the structure back to plumb.

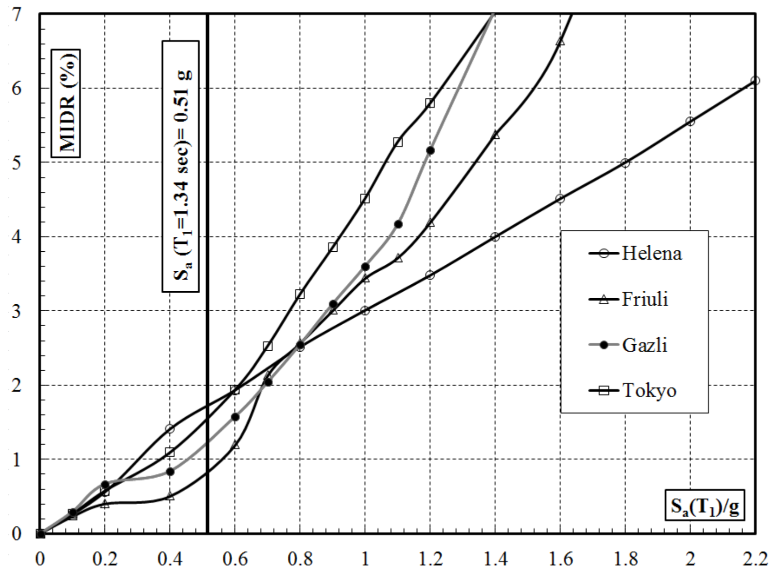


Figure 5.4. Maximum Interstorey Drift Ratio versus spectral acceleration

Table 5.4 Dissipated seismic energy and equivalent corresponding cumulated plastic excursion of dissipative devices for Helena record scaled to $S_a(T_1)$=1.40 g.

Device	Energy Dissipated [kNm]	Equivalent cumulated plastic Excursion [mm]	required stroke [±mm]	Device	Energy Dissipated [kNm]	Equivalent cumulated plastic Excursion [mm]	required stroke [±mm]
D1	45.26	68.07	15.64	D25	539.59	811.41	34.68
D2	20.8	31.27	18.26	D26	53.34	80.21	37.48
D3	20.8	31.28	15.48	D27	54.45	81.88	34.86
D4	20.97	31.53	18.37	D28	53.74	80.81	37.75
D5	20.66	31.06	15.48	D29	54.03	81.24	34.95
D6	21.28	32	18.65	D30	54.01	81.22	37.94
D7	196.93	296.13	24.69	D31	419.53	630.87	30.04
D8	40.87	61.46	27.55	D32	43.68	65.68	32.85
D9	42.58	64.03	24.82	D33	44.78	67.34	30.26
D10	41.25	62.04	27.71	D34	44.02	66.19	33.15
D11	42.2	63.45	24.83	D35	44.57	67.03	30.35
D12	41.47	62.36	27.75	D36	44.23	66.51	33.33
D13	338.96	509.71	32.11	D37	449.05	675.26	26.03
D14	51.46	77.38	34.83	D38	33.83	50.87	28.57
D15	53.53	80.5	32.13	D39	35.58	53.5	26.01
D16	51.92	78.07	35.01	D40	34.21	51.45	28.9
D17	53.06	79.79	32.17	D41	35.36	53.17	26.12
D18	52.24	78.55	35.25	D42	34.62	52.06	29.37
D19	468.03	703.8	36.97	D43	102.63	154.33	16.23
D20	57.81	86.93	39.5	D44	15.08	22.67	17.85
D21	59.58	89.6	36.84	D45	16.92	25.45	15.33
D22	58.21	87.54	39.73	D46	15.26	22.95	18.2
D23	59.17	88.98	36.92	D47	16.7	25.12	15.42
D24	58.6	88.11	40.18	D48	16.23	24.41	19.59

6. CONCLUSIONS

In this paper a design methodology aimed at the failure mode control of Dissipative Truss Moment Frames has been applied with reference to an eight storey building. The main feature of the analysed structural typology is due to the innovative use of dissipative devices located at the bottom chord level of truss girder ends. Therefore, DTMFs are essentially truss moment frames where truss girders are equipped with dissipative (friction or hysteretic) devices whose aim is the dissipation of the earthquake input energy. The combination of this structural typology with a rigorous design procedure for plastic mechanism control allows the design of structures where, under severe seismic events, the primary structural members are free of damage. In fact, the seismic energy dissipation is exclusively concentrated in the specifically located dissipative devices.
Non-linear static analyses have been carried out with reference to the designed eight storey building aiming to provide the criteria to design the stroke of the dissipative devices. To this scope, the Capacity Spectrum Method has been applied. Successively, by performing non-linear dynamic analyses, the effectiveness of the design methodology has been demonstrated.
For sake of shortness, only the results dealing with the seismic response of an eight storey DTMF have been discussed in this paper. However, the preliminary results of incremental dynamic analyses herein presented and those performed with reference to other schemes have pointed out the accuracy of the design methodology, because all the dissipative devices (friction or hysteretic devices) are significantly involved in the seismic energy dissipation without any involvement of the main structure.
The value of the maximum required drift angle (obtained for Reggio Calabria site) is less than the value of the local ductility used in the application of the design methodology for plastic mechanism control (equal to 0.04 rad) and, therefore, it is compatible with the assumed column plastic rotation.
Finally, the obtained results show that the CSM provide a useful tool to design the stroke of the dissipative devices.

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