

NUMERICAL INVESTIGATION OF DISSIPATIVE BEHAVIOR OF CONNECTION USING POST-INSTALLED ANCHORS

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ABSTRACT

Steel to concrete connections using post-installed anchors are nowadays used in seismic-prone countries worldwide. In the case of structures where steel to concrete connections link primarily members (e.g. resistant to seismic loads) or they are used for seismic retrofit, some questions may arise:(a) what is the stiffness of the structure; (b) what is q factor in case of new building (essential in commonly adopted design with response spectra). A lack of knowledge is recognized in the contribution of the connection to the seismic performance at whole structural level. Ductile behavior and hysteretic energy dissipation of the connection become fundamental aspects to be investigated complying with capacity design approach, namely to ensure local dissipation (plastic hinges) preserving the structure as a whole. The present paper addresses the problem of column-to-foundation connection using bonded anchors with stretch length, subjected to seismic loads. Numerical models, using simple beam elements with lumped plasticity, are calibrated starting from experimental results on the sub-structure namely (i) steel column, (ii) steel base plate; (iii) anchors. Changes in anchors' non-linear behavior are investigated. An application to one-story frame subjected to seismic artificial accelerograms is then presented.

Keywords: anchors; seismic design; column-baseplate connections;

1. INTRODUCTION

1.1 Background

The dissipative behavior of steel-to-concrete connection has been extensively studied in the last decade in the steel researcher's community, with particular address to column-to-foundation (indiscriminately called column base) connection. In these work, a lot amount of energies were spent in the investigation of dissipative source due to the plastic behavior of steel plate [see for instance Astaneh-asl (2008); Ermopoulos and Stamatopoulos (1996); Midorikawa *et al.* (2008); Thambiratnam and Paramasivam (1986)]. For design purposes, widely used in the US is the guideline published by AISC (American Institute of Steel construction [Fisher (2006)]), which is based on design static calculation method. Kanvinde *et al.* (2012) developed an analytical solution for the evaluation of the rotational stiffness with an extended version of the component method.

Li *et al.* (2016) have performed numerical analysis on base plate connection using solid finite elements, results have been compared with EC3 simplified calculation based on the standard component method; it was found that Eurocode 3 over-predicts the initial stiffness and bending moment capacity of the connection. This overprediction depends on the design details of steel column-baseplate connection, especially on the arrangement of anchor bolts.

To date, few works coming from specialized research field of anchorages to concrete address the possibility to develop a ductile behavior looking for anchors non-linear response.

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Trautner *et al.* (2016) tested different solution of column-to-foundation connections with post-installed anchors with the introduction of stretch length, tests were limited to a single column size and to connections where the inelastic behavior of the connection was dominated by the yielding of the anchors. The results can be summarized as follow: (i) cast-in anchors shows rotational capacity up to 140 mrad, adhesive anchor up to 90 mrad, undercut anchors up to 60 mrad (as for the investigated geometry); (ii) increased stretch length was associated with only moderate increases in connection rotation capacity, i.e. an increase in connection rotation capacity of approximately of 30% resulted by passing from 3.5 to 8 diameters; (iii) limited testing indicates that the method used to level the base- plate may significantly change the behavior of the connection.

Trautner *et al.* (2017) focused on tensile properties of all-thread and headed anchors, particularly characterizing the ductile behavior. The tensile strength and deformation capacity of commonly used steel grade in the US (according to ASTM standards) were investigated by means of tensile tests. An analytic approach was then developed to calculate the requirement on stretch length based on (a) fundamental period of the structure; (b) anchors arrangement and mechanical properties. A simplified version of the approach for frame buildings, directly relates the elongations demand to the drift ratio and anchor lever arm, assuming rigid plate condition.

Muciaccia (2017) reviewed the criteria on which current European design codes dealing with anchorages treat the seismic design of steel-to-concrete connection. Capacity design method is recalled for all the possible scenarios, i.e. protection of the fasteners and protection of the attached elements. An experimental investigation on column base was carried out reproducing the design conditions. The present paper reviews part of the experimental campaign in the first section.

Hutchinson (2017) aimed the anchor yielding in a single bay steel prototype frame tested at UCSD having a rocking type behavior as a result from column up-lift, the super-structure was slightly unloaded due to the rigid-body displacement field characterizing rocking. In particular, a reduction of 25% of base shear with respect to the fixed base structure was pointed out.

1.2 Summary and scope of the work

The present paper addresses the seismic behavior column-to-foundation connection using post-installed anchors. Reference is made to the experimental campaign carried out by Muciaccia (2017) in which bonded anchors were used, due to their versatility to reach higher pullout capacity by increasing the bonded length.

Numerical models using simple beam elements with lumped plasticity are calibrated starting from experimental results on the sub-structure. The model consists of a steel column (IPE 240 beam elements) attached to an end plate (modeled with rigid links), bonded anchors are modeled as beam elements assuming a linear-elastic bond-slip law. In this phase load cases are conventional, i.e. monotonic test in displacement control and displacement cyclic protocols. Non-linear dynamic analyses with time history integration are carried out and structural response is compared with experimental data.

Key points addressed by further numerical tests are (i) changes in anchors behavior (tension/compression, tension only); (ii) reduced compression capacity to simulate instability phenomena.

An application to a simple one-story frame subjected to seismic artificial accelerograms is then presented. The example shows the structural performance assuming the column base connection undergoing in non-linear response, for different values of the behavior factor (q-factor).

2. EXPERIMENTAL RESULTS

An experimental on column-to-foundation connection investigation was carried out by Muciaccia (2017). Here test setup and procedure are briefly reviewed, further details can be found in the original paper. In Figure 1 the layout of the test setup is reported. A steel column (IPE 240) was connected to a concrete slab (C20/25 - uncracked) by post-installed anchors. The slab was fixed to the strong floor by a tie-down system.

The column was laterally loaded by a hydraulic cylinder (100 kN capacity, double stage) placed on a side contrast frame. The distance between the direction of application of the load and the concrete surface was selected in order to apply the desired pair of shear and bending moment to the fastening. In the specific case, 1.50 m was the lever arm between the actuator and the concrete surface. Both monotonic and cyclic tests were performed in displacement control, and the actuator displacement was assumed as the control signal.

Connections types varied for different design configurations: (i) protection of the attached element; (ii) protection of the fasteners. For the purpose of the present discussion, only the first case will be considered, focusing on dissipation source due to the stretch length of the anchor. In Table 1 the considered configurations are summarized. The attached element (IPE 240) and the embedment depth for the bonded anchor ($h_{ef} = 310$ mm) were adopted in both cases since they encompass the capacity design approach, i.e. promoting the steel yielding of the threaded rod (M16 [10.9]). Base shear was reacted by shear key solely (M30).

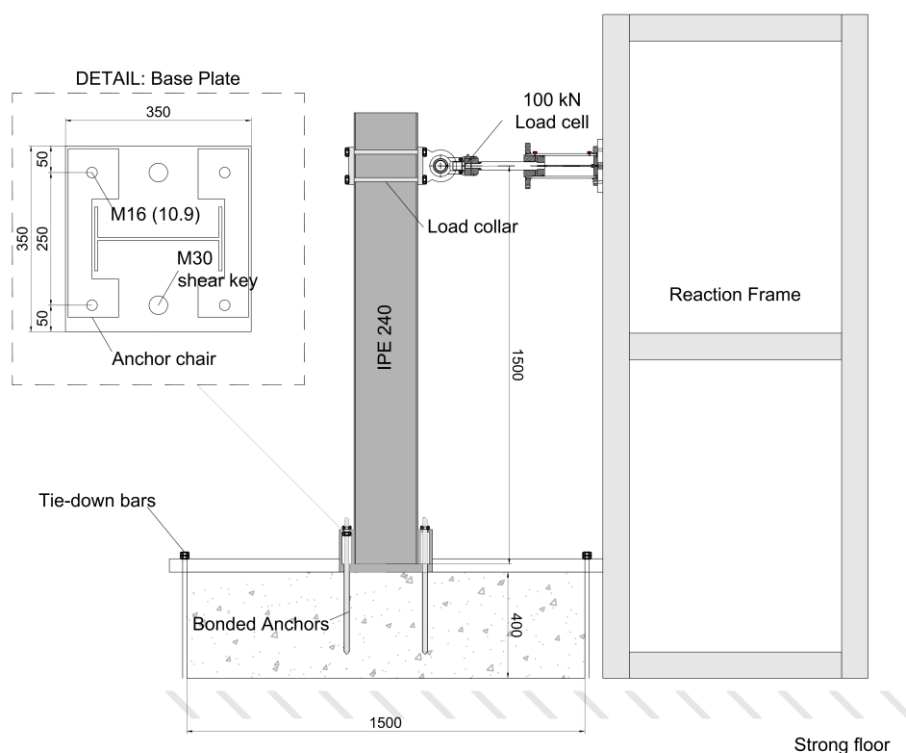


Figure 1. Experimental setup column-to-foundation in Muciaccia (2017)

Table 1. Considered connections in Muciaccia (2017) work

Case	Anchors	Steel column	Steel Plate	Stretch length
Case 2	4 x Bonded ($h_{eff} = 310$ mm)	IPE 240 (S275)	350 x 350 (S275)	not present
Case 3	4 x Bonded ($h_{eff} = 310$ mm)	IPE 240 (S275)	350 x 350 (S275)	130 mm

3. NUMERICAL MODELING

The test data were used to evaluate the predictive capability of a simple three-dimensional numerical model carried out in SAP 2000. The aim of the model is to catch the hysteretic behavior of the connection performing non-linear dynamic analyses.

3.1 Model Description

The model includes beam elements only, this is thought to maintain simple and computationally efficient the modeling phase which could be included in a design procedure. In the following, the assumed representation of the column-to-foundation under cyclic loading test is detailed, describing the components and computational method. Further details are shown in Figure 2.

3.1.1 Column

Beam elements (IPE 240 [S275]) in linear-elastic regime. Tip node is restrained against displacements, while rotations are free. Settlement of horizontal displacement and the associated reaction force is assumed as a constitutive law of the beam-to-column to be compared with experimental results previously resumed.

The bottom node transfers displacement and rotations the steel plate modeled by rigid link elements

3.1.2 Steel plate

Rigid link element connects the central node (fully shared with the column) to anchor tip nodes. Each node is the vertex of a triangular gridwork, which ensures enough stiffness in transfers displacements without the need to enforce kinematic constraints.

A smeared spring support (compression-only) is implemented to account for the concrete base reaction, stiffness coefficient 100 kN/mm^2 is assumed tuned for numerical ill-conditioning prevention.

3.1.3 Anchors

Anchors are modelled as beam elements with lumped plasticity hinge (PH), having different lengths. The cross-section is circular with 16 mm diameter. The tip node is connected to the steel plate, while the bottom node is restrained against displacements in two horizontal directions. In the vertical direction, an elastic spring restrains the displacement, accounting for slip behavior of bonded anchor. The elastic coefficient of $k_a = 140 \text{ kN/mm}$ is assumed with reference to experimental results on single anchor pullout test.

Compression of the anchor is prevented via stiffness penalty except for the case of stretch length with double nuts.

3.1.4 Plastic hinge

All the non-linear source for the column-to-foundation details are stored in the anchors as lumped plasticity hinges (PH) acting for axial force and associated axial displacement.

PH formulation is based on the definition of the backbone curve, the hysteresis model, and the hinge length. Such parameters were calibrated with reference to experimental results obtained for the whole sub-structure, i.e. column-to-foundation connection.

The backbone curve is defined in stress-strain space assuming the following considerations: (a) elastic-plastic model with hardening for the tension half-space, having $20 \cdot \epsilon_y$ as elongation in the hardening phase is consistent with results founded in Trautner *et al.* (2017); (b) elastic-perfect plastic model in compression for the compression half-space, assuming a reduced yielding stress and abrupt resistance lost to account for instability phenomena.

The stress-strain relation is automatically converted into a force-displacement relation scaling for the cross-section area and the hinge length respectively. For the last, the entire length of the anchor beam element is assumed, meaning that plasticity phenomena are spread all over the length.

The cyclic model for force-displacement is based on the model proposed by Takeda (1970). Even not specifically developed for steel details the model can be assumed with a discrete level of approximation.

3.1.5 Displacement History (load case)

A displacement time history is applied at the top node of the column according to the test procedure carried out by Muciaccia (2017). The cyclic period $T = 100$ sec is assumed in order to keep the motion in quasi-static condition, differences with respect the experimental results are solved by scaling procedure.

3.1.6 Solution method

The numerical integration of the motion is performed by using Newmark method with $\beta = 0.25$ and $\gamma = 0.5$ (mean acceleration method). Consistent - Mass matrix is assumed.

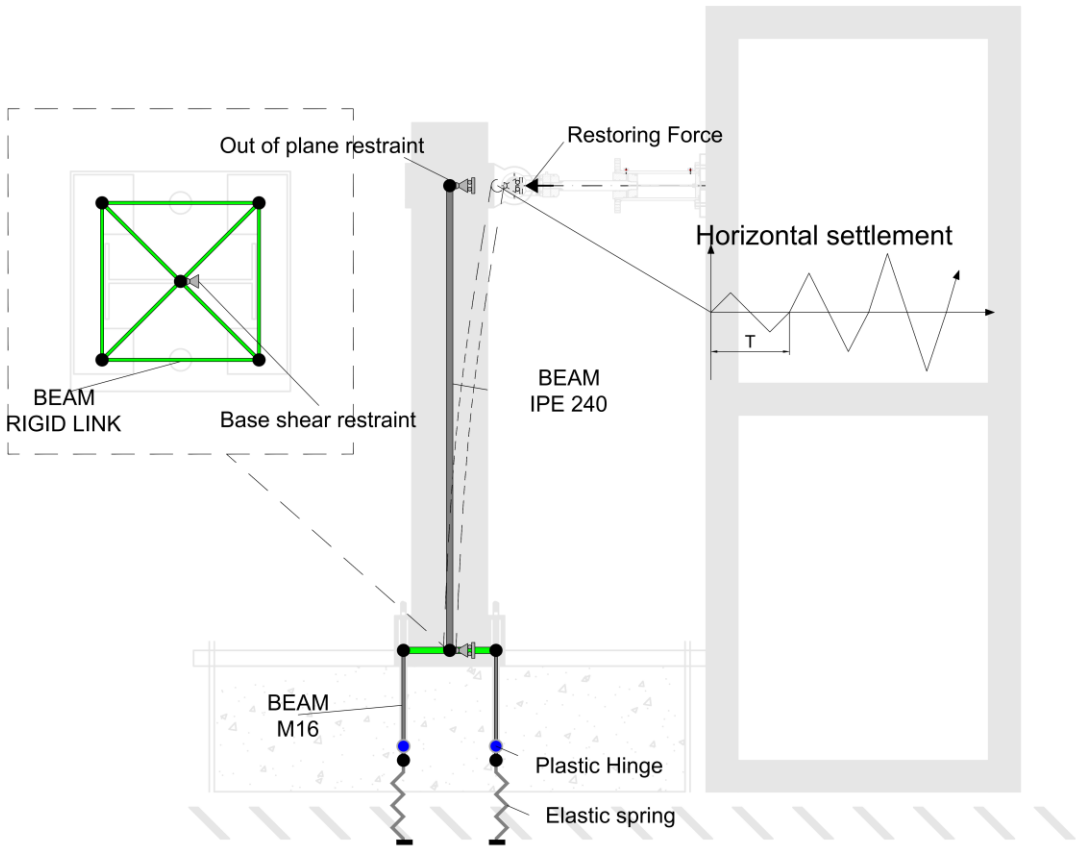


Figure 2. Numerical modeling for column-to-foundation connection

3.2 Calibration of the model

As mentioned in the previous descriptions, parameters of interest in calibration are (i) yielding strain and ultimate strain for the backbone curve (symmetric case); (ii) reduction of compressive yielding stress due to instability phenomena for non-symmetric backbone curve. In the following sub-sections, numerical models are described with reference to load-displacement plot (settlement of the column’s tip node and restoring force) and cumulated plastic displacement (axial displacement in anchors).

3.2.1 No stretch length

In Figure 3 results are shown for the case of no-stretch length, i.e. case 2 in Table 1.

The anchor unrecovered displacement after unloading is consistent with numerical result. Cumulated plastic displacement for the anchor acting in tension is confirmed as the only non-linear source, while bond-slip behavior can be reasonably maintained in elastic regime due to large embedment depth.

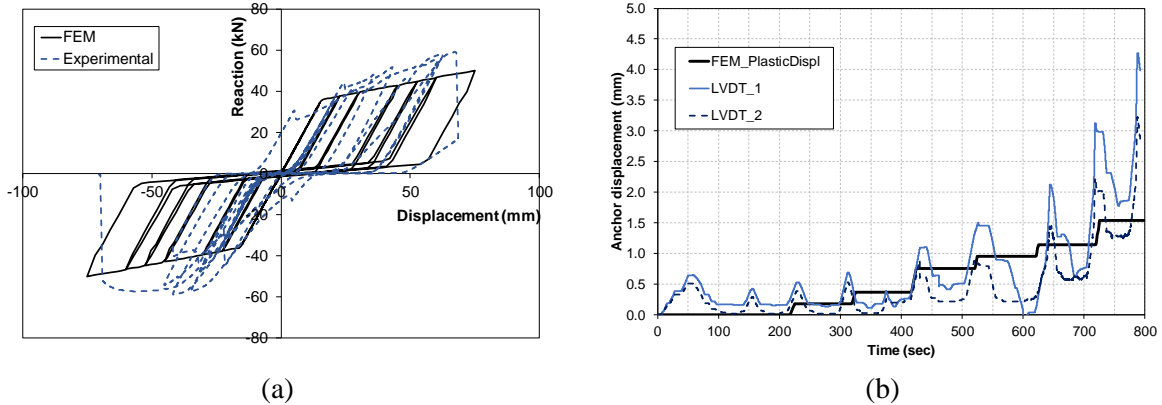


Figure 3. Results for the case of anchors without stretch length: (a) load vs displacement; (b) time variation of displacement

3.2.2 Double nut

In Figure 4 the case 3 (130 mm of stretch length – two nuts) is presented, anchors are modeled assuming a PH with the symmetric definition in tension and compression. Numerical model fits quite well the hysteretic behavior at least for the first cycles. During last cycles, compressed anchors undergo in instability phenomena, hence further investigation is needed.

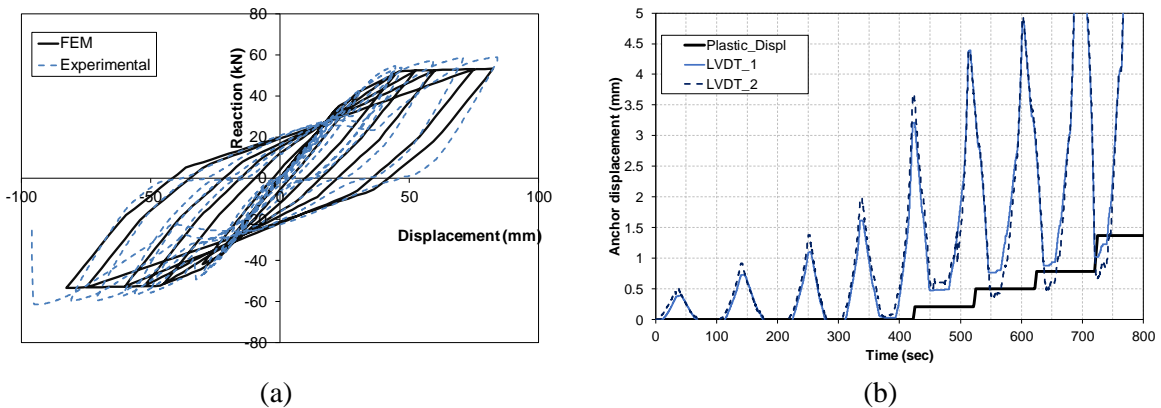


Figure 4. Results for the case of anchors with stretch length [double nuts condition]: (a) load vs displacement; (b) time variation of displacement

3.2.3 Reduced compression

Reduced compression performance due to buckling is pursued by adopting a non-linear static analysis with assuming a non-symmetric definition for the PH. In the compression half-space, the yield stress was assumed $0.5 f_y$, being f_y the nominal yielding stress of the bars (steel class 10.9). This approach assumes theoretical instability curves proposed by Ziemian (2010).

Results are shown in Figure 5 (a). Target response was the first cycle presenting instability phenomena in experimental test, considering the non-zero load at null displacement (due to re-loading after displacement accumulation) not relevant.

In the first branch anchors act in tension and compression, as the anchors fail in compression a snap down in load occurs; then increasing the displacement, compressive reaction offered by rigid plate comes into place and a new neutral axis is formed as equilibrium balance between concrete compression and tension in anchors. The proceeding loading path is characterized by hardening in stretched anchors.

This mechanism explains the possible drawback in using the anchors in compression without taking care of instability phenomena (see the experimental evidence in Figure 5 [b]), i.e. after buckling the connection can be treated as a single nut having lost the anchor compressive reaction (see the following sub-section for details). Further investigations are needed in order to predict the behavior under cycles.

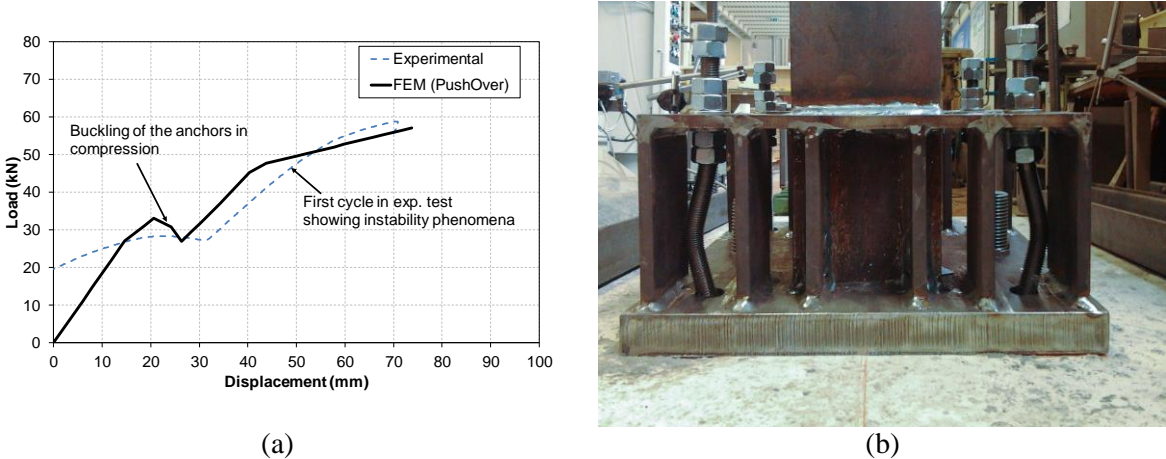


Figure 5. Results for the case of reduced compression: (a) load vs displacement; (b) instability phenomena of compressed anchors in Muciaccia (2017)

3.2.4 Use of single-nut

A modified model was used to predict the behavior in single-nut condition (see for instance the work done by Trautner *et al.* (2016)). The modification consists in eliminating the compression reaction of the anchors maintaining the stretch length. Results are shown in Figure 6.

The connection exhibits a pinching behavior, meaning that the re-centering after the unloading path, is effected without restoring force. Such behavior even if unavoidable in case of simple up-lift column systems badly contributes to damage. The restoring force applies as in impact load indeed, and connection components might be overstressed due to dynamic amplification as noted by Tremblay and Filiatrault (1996) for tension-only braced frame systems.

As one might note the displacement plastic cumulation of deformed anchors in tension increases with respect to the case of double nut. Such difference can be explained by looking at Figure 7. The same amount of rotation is imposed to the two system modeled as done so far by beam elements.

The amount of the imposed rotation is such that a plastic deformation increment occurs in anchors. In the system with double nuts, anchors (active both in tension and compression) will be deformed by the same quantity with a different sign, while in single nut condition anchors in tension are deformed by the double. Two approximations are here invoked: steel base plate behaves rigidly and concrete compressive deformation is neglected.

Summarizing:

- Assuming anchors active both in tension and compression (double nut condition) may be beneficial in terms of reduction in the plastic cumulation of displacement (or base rotation). However, instability phenomena should be prevented including explicitly those in the calculation of the stretch length. If buckling occurs the connection behaves in tension-only.
- Tension-only (single nut condition) leads to a pinching hysteretic behavior and damage (evaluated as cumulative plastic displacement/rotation) is higher.

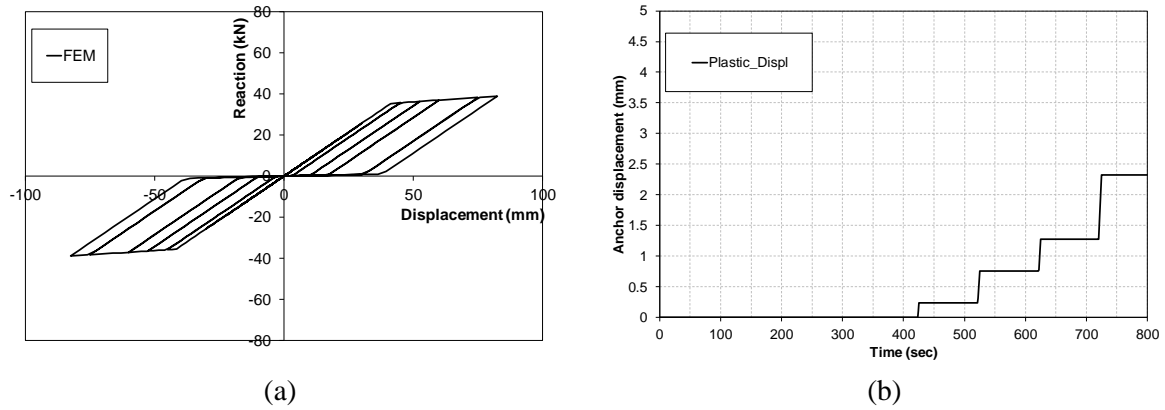


Figure 6. Results for the case of single nut condition: (a) load vs displacement; (b) time variation of displacement

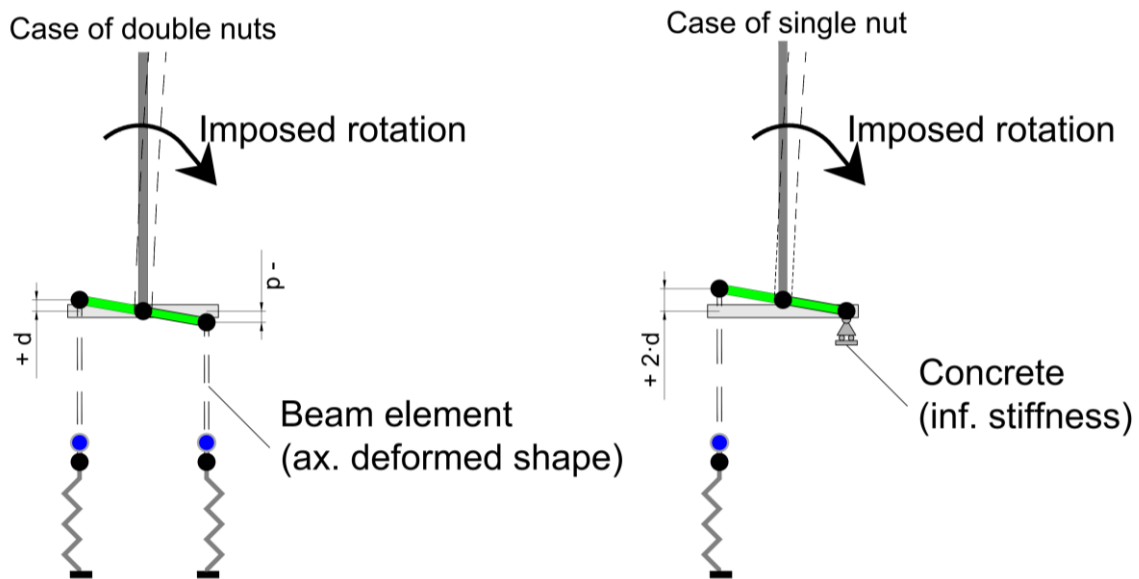


Figure 7. Different kinematic mechanisms in plastic stage for double nut condition (left), single nut condition (right)

4. APPLICATION TO ONE-BAY PORTAL FRAME UNDER SEISMIC EXCITATION

4.1 Description of the studied system

4.1.1 Design criteria

A simple portal frame structure is addressed to evaluate the seismic performance of the base column connections, described in the previous section. Specifically, the conditions of double nuts [3.0] and single nut [3.1] are investigated.

In Figure 8 (a), overall dimension and details of modeling for the one-bay and one-story frame are depicted. The frame is intended to represent a steel moment-resisting frame in which only the base connections undergo in non-linear behavior under seismic action. In doing this capacity design applies as follow:

- (1) Protection of the column (IPE 240) is applied at the base joint according to Muciaccia (2017)
- (2) Transverse beam assumes the same cross-section of the column (i.e. IPE 240), thus the elastic behavior of the superstructure is pursued. Moreover, the beam-to-column connection is modeled as perfectly rigid.

The overall mass is assumed in order to keep the period of the structure in the range 0.20 – 0.30 sec.

In this case, assuming the EC8 acceleration spectrum (Type B), the most spectral amplification is obtained. The resulting mass is then split and lumped at the beam-to-column joints.

4.1.2 Dynamic modeling and conditions for non-linear analysis

Dynamic analyses have been performed for different values of the design q-factor value. In the long and intermediate period range, a linear law relates the peak-ground acceleration (PGA) and the q-factor, as remarked by Martinelli *et al.* (1998) and Perotti (1996). Firstly, an artificial accelerogram is generated having the elastic spectrum (Type B and $PGA = 0.525 \cdot g$) as a target (see Figure 8 [b]). The structure behaves elastically when subjected to the generated accelerogram (motion of the restraints in one direction), i.e. $q = 1$. The structural response is computed via Newmark method assuming the modeling features described in the previous section. Then using the linear relation between PGA and q as previously mentioned, the accelerogram is scaled assuming for the q-factor a range $2 \leq q \leq 5$; the non-linear dynamic analysis is repeated for all of the scaled accelerograms.

4.1.3 Response parameter

The peak response parameters to be used to judge the seismic performance has been defined as the mean extreme value (μ) and the standard deviation (σ) of the drift ratio. This, as usual, is defined as the ratio between the tip node displacement and the column's height. Plastic tensile strains for anchors have been selected as a measure of structural damage.

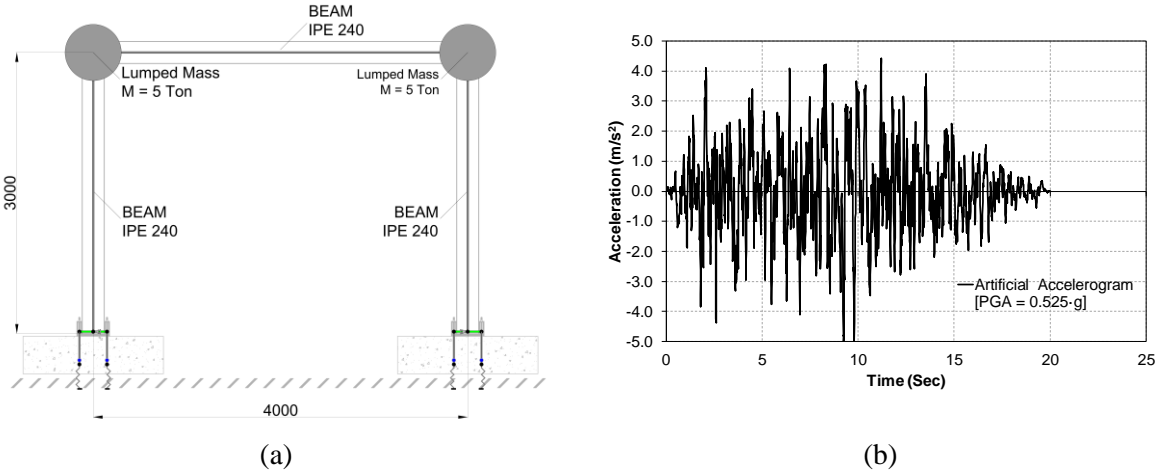


Figure 8. One-storey frame example: (a) modeling layout; (b) artificial accelerogram generated from elastic spectrum EC8 [Type B and $PGA = 0.525 \cdot g$]

4.2 Results of the numerical analyses

The main results obtained from the numerical analysis will be described in this section. Two different conditions for the base plate connection were considered: [3.0] anchors are beam elements with 130 mm of stretch length, they act both tension and compression (results to sub-component cyclic test were presented in Figure 4); [3.1] anchors with same stretch length act in tension only.

In Figure 9 the drift ratio time history for the two system is presented for the extremes values of the behavior factor range considered, i.e. $q = 2$ and 5. System 3.0 shows an increase in the peaks response preserving the natural period of the structure as the q -factor increases. While system 3.1 shows a less deformable response but the structural period is increased with the q -factor.

In Figure 10 the different behavior of the studied systems can be inferred. In (a) the $\mu + \sigma$ of the peaks values for the drift ratio versus the q -factor are shown. The tendency of system 3.0 to present much higher degree of deformability is confirmed. In (b) cumulative plastic strains indicate, for system 3.1, a pronounced increase in damage starting from the value $q = 3$. This behavior reflects what was already pointed out for cyclic tests under quasi-static condition. Moreover, it might be concluded that if the level of plastic strain above 20% is not reliable, system 3.1 should assume q -factor values less than three.

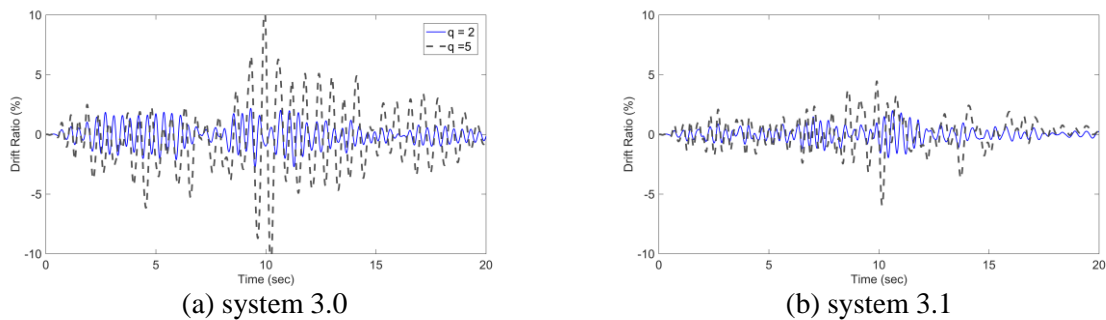


Figure 9. Drift ratio time response: (a) double nuts condition [3.0]; (b) single nut condition [3.1]

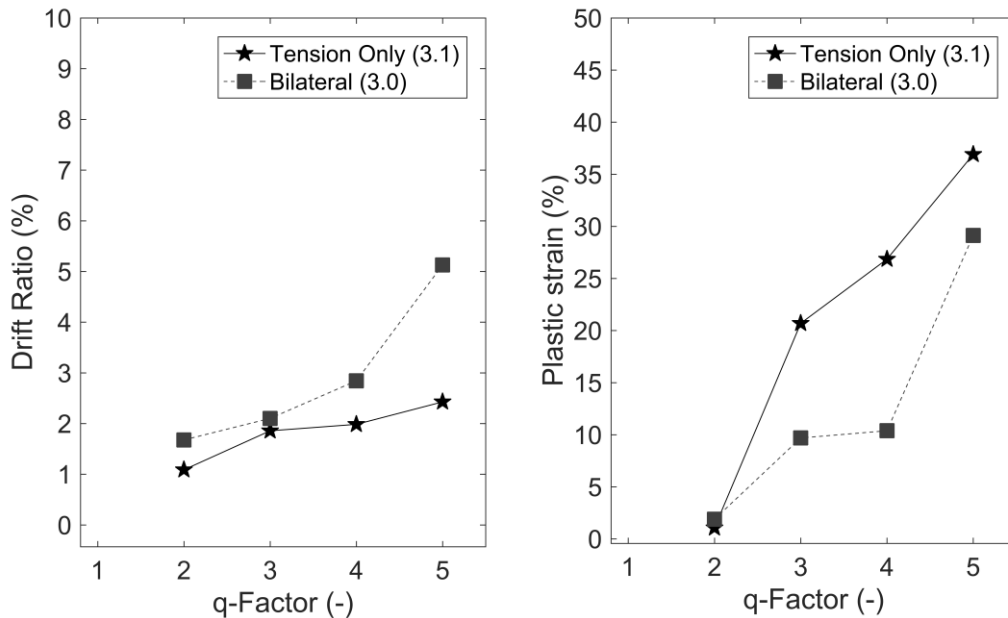


Figure 10. Evaluation of the structural performance under seismic action: (left) drift ratio [$\mu + \sigma$] vs q -factor; (right) plastic deformation vs q -factor

5. OUTLOOK

Further investigations are needed to include instability phenomena for compressed anchors in dynamic non-linear analyses: a change in the definition of plastic hinge might take into account a second-order bending effect and its interaction with plastic evolution.

A more refined constitutive law for steel material has to be involved to account for the Bauschinger effect, i.e. the hardening behavior might be affected by the strain history.

Definition of q-factor might be extended to a more complex moment resisting frame or braced systems in which capacity design rules apply.

6. CONCLUSIONS

The present work addressed the problem of column-to-foundation connection using post-installed bonded anchors under seismic actions. The use of stretch length in improving the performance of the connections was considered, and different conditions for anchor compression reaction were compared: (i) double nuts [tension/compression]; (ii) single nut [tension only]. The results are extended to a simple structural application.

The following conclusions can be drawn:

- Numerical models including beam elements only with lumped plasticity maintain simple and computationally efficient the modeling phase and they could be included in a design procedure.
- Nonlinearities are concentrated in the plastic behavior of steel under axial loads and the slip of the adhesive was modeled with an equivalent linear-elastic law.
- Plastic hinges for the axial behavior of the anchors were defined by stress-strain envelope (backbone curve) assuming an elastic-plastic law with isotropic hardening, Takeda hysteretic model was then applied for cyclic response. The ultimate strain was tuned to $20 \cdot \varepsilon_y$ after the calibration process.
- Cyclic behavior of the connection, experimentally investigated in quasi-static condition can be reproduced quite accurately by non-linear dynamic analysis. The occurrence of instability phenomena for compressed anchors can be replicated by assuming a reduced compression backbone. In this case, the stress threshold was assumed as equal to $0.5 \cdot f_y$ with a limited post-yield branch.
- Assuming anchors active both in tension and compression (double nut condition) may be beneficial in terms of reduction in the plastic cumulation of displacement (or base rotation). However, instability phenomena should be prevented including explicitly those in the calculation of the stretch length. If buckling occurs the connection behaves in tension-only condition.
- Tension-only (single nut condition) leads to a pinching hysteretic behavior and damage (evaluated as cumulative plastic displacement/rotation) is higher.
- In a simple structural application (one-bay and one-story frame) q-factor values higher than 3 are achievable only for tension/compression case, due to reliable cumulative plastic displacement.

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