Deliverable 10.3

Design and application guidelines for Stick-slip and Hysteretic dissipative anchors with embedded sensors

Due date: December 2012

Submission date: December 2012

Issued by: ITAM

WORKPACKAGE 10: Guidelines for end-users

Leader: ITAM

PROJECT N°: 244123

ACRONYM: NIKER

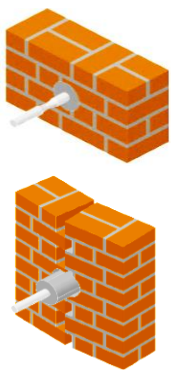
TITLE: **New integrated knowledge based approaches to the protection of cultural heritage from earthquake-induced risk**

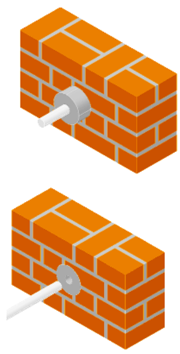
COORDINATOR: Università di Padova (Italy)

START DATE: 01 January 2010 DURATION: 36 months

INSTRUMENT: Collaborative Project
Small or medium scale focused research project

THEME: Environment (including Climate Change)





Dissemination level: PU Rev: FIN

INDEX

1 INTRODUCTION

1.1 DESCRIPTION AND OBJECTIVES OF THE WORK PACKAGE

The exploitation of NIKER results covers both the use of exploitable knowledge and exploitable measures and products. Exploitable knowledge brings mainly non-commercial benefits for project participants and beyond the partnership (e.g. cultural heritage institutions, owners, RTD performers). By means of guidelines prepared in WP10, disseminated as described previously in other deliverables, the internal processes are improved for authorities in charge of CH maintenance. In such a way, cultural institutions and owners of cultural heritage can integrate the technologies and methodologies developed into their project and will be able to carry them out more effectively and more efficiently. The WP10 is subdivided into:

WP10.1: Guidelines for specific problems. These guidelines outline the main results obtained in WP3; WP4; WP5; WP6, WP7 and WP8 and are intended for designers and users of the technologies. Therefore, the entire work carried out into the project will be substantially simplified for the needs of the end-users and designers, by providing simple design rules, design formulations and design charts.

WP10.2: Guidelines for integrated methodologies. These guidelines summarize the main results obtained in WP7; WP8 and WP9 and are mainly oriented to designers or bodies responsible of the management and maintenance of the structures. They will contain the description of the new integrated knowledge based approaches for the protection the CH from earthquake-induced risks emerging from the project.

The main objectives of WP10 can be summarised as follow:

- produce guidelines for the direct end-users of the developed technologies and tools (designers, architects, engineers, construction companies, bodies responsible of building maintenance, etc), with practical information on design of interventions, execution of techniques, assessment tools, monitoring procedures;
- produce guidelines for production and installation of advanced instrumented dissipative devices;
- produce guidelines for owners and end-users of the developed technologies and tools (public entities, bodies responsible of building maintenance, authorities, etc), with description of step-by-step integrated methodology for effective protection of cultural heritage;
- spread awareness and establish reliable, effective, compatible, integrated approaches for the protection of cultural heritage from earthquake-induced risks.

The outcome of Workpackage 10 is presented in 5 deliverables, covering the aspects listed above

1.2 SUMMARY AND OBJECTIVES OF THE DELIVERABLE

Deliverable 10.3 focuses on the implementation of innovative devices for the strengthening and upgrade of structural connection of heritage buildings. Similarly to D10.2, information for the appropriate choice, design and installation of a number of systems is provided; however, in this case the focus is on innovative techniques that rely on dissipation of energy and enhanced ductility.

2 ENERGY-BASED STRENGTHENING SYSTEMS FOR HERITAGE

In the last decades, drawing on the experience deriving from earthquakes such as Northbridge, California, USA, 1994, and Kobe, Japan, 1995, structural engineers have progressively abandoned capacity methods and moved on to performance-based techniques (Priestley, 2000), which focus on the enhancement of the ductility and the use of additional dissipative elements, rather than relying on stiffness and strength for the purpose of improving the seismic behaviour of structures.

In the field of new built, this concept is widely applied (Symans et al., 2008) through the installation of various damping systems: looking at the existing technical literature it is fairly straightforward to find a number of examples regarding prototype development and experimental assessment (Aiken et al., 1993; Morgen and Kurama, 2008; Christopolus et al., 2008), analytical and computational studies (Constantinou and Symans, 1993; Lopez Garcia and Soong, 2002; Lin and Chopra, 2008), recommendation for design and implementation (Rodgers et al., 2008) and application to case studies (Aiken et al., 1998; Chang et al., 2006).

In the field of heritage structures, although the use of ductility and energy-based systems is provided for and encouraged by current codes (EN 1998 Eurocode 8; Italian Ministry of Cultural Heritage and Activities, 2006), traditional stiffness-based systems are still allowed for and widely applied. In fact, innovative systems rarely meet some of the requirements – reversibility, low impact – required for interventions on historic structures. Indeed, few high-profile case studies and research projects appear in literature (Benedetti, 2004; Indirli and Castellano, 2008; Mandara and Mazzolani, 1994).

It would be instead highly beneficial that high-ductility systems were more widely applied, especially to the structural connections of heritage buildings, and this for a number of reasons: firstly, the global seismic behaviour of historic masonry buildings is highly influenced by the integrity of connections among vertical and horizontal structural elements, which ensure the so-called box behaviour. This, providing the transfer of inertial and dynamic actions from elements working in flexure out of plane to elements working in in-plane shear, leads to a global response best suited to the strength capacity of the constitutive materials, and hence enhanced performance and lower damage level. However, strength-based systems are not always able to effectively restore connections due to the inadequate compatibility of the additional elements with the original materials in terms of stiffness and weight. On-site observations collected in historic centres in the aftermath of major earthquakes, such as L'Aquila earthquake, Italy, April 2009 (D'Ayala and Paganoni, 2011) corroborate such hypothesis, showing how heavy elements, like beams, worsen the dynamic response, often leading to tragic collapses.

On the other hand, other traditional systems, such as cross-ties, which have been and are still commonly applied in rehabilitation practice all over Europe (Tomaževič, 1999), meet the requirement of restoring the box-like behaviour without excessively increasing the mass bearing on the original structure. Yet anchors can also cause pull-out damage and punching failure at the head of the anchorage and this is a major problem when damage limitation should be pursued avoiding cracking in precious plasters, frescoes, or other culturally valuable finishes. Not even traditional systems are fully clear of shortcomings.

Ductility-based systems could instead tackle the problem of brittle failures by allowing controlled relative displacements or rotations, limiting the load transferred to the original materials and improving the dissipation of energy at the joint. Accordingly, a set of dissipative devices have been developed and validated by the NIKER consortium's partners; devices are specifically designed taking into account issues and constraints peculiar to heritage structures, such as low intrusiveness, reversibility, ease of maintenance and so forth.

As result of this process, a series of indications for the choice, installation and design of these dissipative devices are presented in the following to the purpose of providing end-users with clear guidance regarding the criteria to be used when deciding to apply a dissipative device for the upgrade of a structural connection in a heritage building. Different types of connections are considered, so as to offer the reader a range of options.

3 DESIGN AND IMPLEMENTATION OF ENERGY-BASED DEVICES FOR THE STRENGTHENING OF HISTORIC STRUCTURAL CONNECTIONS

Drawing on the need for ductility and energy-based systems specifically designed for the strengthening of structural connections of heritage buildings, NIKER partners developed and refined a set of techniques ad hoc.

In the following the recommendations for correct choice, installation and design of such systems are reported. Prescriptions should be used as guidance when approaching the problem of performing a strengthening intervention that might be compatible with the original materials and in line with the requirements of design codes such as Eurocode 8 (EN 1998), which advice in favour of high ductility rather than high strength.

Design procedures draw on the same criteria as those reported in D10.2: strengthening systems are divided into sub-components to which one type of failure controlled by a single parameter can be associated. The whole set of parameters determines the global response of the system, as each identifies a capacity and can be correlated to an analytical model, thus allowing the dimensioning of the strengthening system according to a hierarchical process of components' failure. Individual components and system capacities can be calculated through the formulae prescribed by design codes, whereas input values are derived from tests or, alternatively, from producers' specifications and code requirements, if any.

Tables including the meaningful performance parameters for each system, including experimental data and references to codes are reported in the following.

For a more detailed description of the design process, the reader should refer to D10.3.

It is worth highlighting that the strengthening systems described below are still under development, and as such end users should be particularly careful when deciding to use them: each application is indeed highly specific and the design of the whole strengthening system should be thoroughly considered and verified.

ENERGY-BASED DEVICES FOR CONNECTIONS					
INTERVENTION		APPLICATION			
Ref. Section	**Type**	**Component**	**Material**	**Prevented failure mechanism**	**Repaired damage / Improved performance**
3.1	Hysteretic/ frictional anchoring devices w/o monitoring and early warning system	Wall-to-wall connections	Brickwork masonry	Detachment of orthogonal walls, out-of-plane overturning of panel perpendicular to motion direction and brittle failures at head of anchorage (e.g. punching, pull-out)	- Controlled relative displacements at the joint of the two walls; - Prevention of damage and cracking in the parent material and anchor assembly - Reduction of load transmitted to parent material; - Enhanced dissipation of energy; - Possibility of monitoring evolution of damage and response to pseudo-static and dynamic phenomena - Possibility to trigger early warning.
3.2	Stick-and-slip carpentry connection	Roof carpentry connections	Timber	Roof frame deformation during horizontal wind or earthquake loads	-Improved energy dissipation
3.3	Ductile anchors	Wall-to-wall and wall-to-floor connections	Rubble stone masonry and timber elements	Detachment of orthogonal walls, out-of-plane overturning of panel perpendicular to motion direction and brittle failures at head of anchorage (e.g. punching, pull-out)	- Prevention of damage and cracking in the parent material - Improved dissipation of energy;

3.1 DISSIPATIVE ANCHORING DEVICES W/O MONITORING

Hysteretic anchoring device		
Design parameters	*Applicability*	*Advantages and limts*
- Yielding strength of dissipative element of device; - Tensile strength of metallic anchor rod; - Bond strength between anchor rod and binder; - Bond strength between binder and parent material; - Tensile strength of parent material; - Bond strength at mortar joints.	Generally applicable. In case of particularly weak or loose substrata, the performance of the strengthening system is improved by grouting of parent material or anchor undercutting.	*Advantages* - Negligible increase in mass; - No aesthetic impact; - Reversibility; - Prevention of brittle failures typical of standard anchors; - Prevention of damage in historic substratum - Various level of performance that can be tailored to design limit states; - Replaceable dissipative element. *Limits* - Presence of precious finishes and geometry of the building might restrict prevent the possibility of drilling anchors in the required position; - If oversized, brittle failures typical of standard anchors might occur.
Frictional anchoring device		
Design parameters	*Applicability*	*Advantages and limts*
- Slip load of frictional device, this being controlled by the coefficient of friction and by the applied perpendicular pressure; - Shear/bending capacity of stops of frictional plates; - Tensile strength of metallic anchor rod; - Bond strength between anchor rod and binder; - Bond strength between binder and parent material; - Tensile strength of parent material; - Bond strength at mortar joints.	Generally applicable. In case of particularly weak or loose substrata, the performance of the strengthening system is improved by grouting of parent material or anchor undercutting.	*Advantages* - Negligible increase in mass; - No aesthetic impact; - Reversibility; - Prevention of brittle failures typical of standard anchors; - Prevention of damage in historic substratum - Reduction of load transmitted to substratum; - Various level of performance that can be tailored to design limit states; - Replaceable dissipative element; - Flexible performance that can be match to substratum mechanical properties. *Limits* - Presence of precious finishes and geometry of the building might restrict prevent the possibility of drilling anchors in the required position.

Monitoring anchor device		
Design parameters	*Applicability*	*Advantages and limts*
- As dissipative anchoring devices + - Damage mechanisms to be monitored → number and types of sensor and design of data logger.	- Generally applicable. In case of weak or loose substrata, performance improved by grouting of parent material or anchor undercutting; - Continuous electrical supply required; - Use of UPS recommended; - Internet connection required in case of remote control; - Data logger must be positioned indoor, nearby the anchor; - Recess might need cutting in parent material to run cables from anchor to data logger.	As dissipative anchoring devices + *Advantages* - Joint strengthening and monitoring system; - In-house assembled → quick installation to the advantage of safety for people working on-site; - Monitoring of localised phenomena; - Compact and simplified monitoring. *Limits* - Power supply and internet connection required; - Speed of data acquisition and processing.

Definition and scope

Drawing on the observations already reported in Chapter 2, the University of Bath and Cintec International Ltd jointly developed (Paganoni and D'Ayala, 2009 and 2010a-b; Paganoni et al., 2010) a dissipative device for the passive protection of heritage buildings in seismic prone areas to the purpose of overcoming the drawbacks of standard metallic anchors.

Similarly to metallic cross-ties, the anchoring devices aim to:

- restore the unitary behaviour of a structure by ensuring the connection between sets of perpendicular walls;
- reduce the risk of out-of-plane mechanisms of masonry panels.

Additionally, the devices also aim at:

- preventing brittle failures at the head of the anchorage, such as punching and pull-out, which normally affect metallic anchor, both in the set-up with end plate or fully grouted;
- allowing relative controlled displacements between two walls, thus ensuring ductility of the connection and dissipation of energy within the standard drift limits prescribed by codes;
- reducing the load transmitted to the weak substratum by the anchorage.

These goals are reached by means of either a stainless steel element, shaped to optimise its post-elastic behaviour, or a device relying on a friction mechanism set to be triggered for a certain level of pulling/pushing force; the devices are placed in series with a metallic grouted anchor, in correspondence of an existing crack, or where damage is most likely to occur as consequence of the poor quality of connections or simply of the wear and tear of the structure.

Applicability conditions

The system is generally applicable and provides better performance than standard anchor thanks to its ductility. This means that in case of good quality parent material, the anchoring devices will improve the performance in respect to a standard anchor, reducing or even eliminating damage in the parent material and in the anchor assembly. In case of very poor or loose parent material, the dissipative anchoring device will ensure a higher level of safety in respect to standard anchors, which might instead pull out without offering any resistance to the seismic action.

If one wants to further improve the performance level in weak substrata, grouting might be performed to the purpose of facilitating the transmission of load from the anchor to the parent material. Undercutting of the drilling hole can also be applied to achieve better pull-out capacity of the anchor assembly.

Furthermore, in case of weak parent material, it is recommended that the frictional device, rather than the hysteretic device, is used as it allows a more refined control over the activation slip-load and hence on the performance of the system.

Design

Demand in terms of tensile force on metallic anchors is calculated as:

$$F_D = Ma_i \quad (1)$$

where:

- *M*: mass of structure that bears on the ith anchor of the strengthening system. It depends on the geometry and construction arrangement of the building, including horizontal structures, and on the lay-out of the set of anchors to be designed;
- a_i: horizontal acceleration experienced by the mass M. An estimate of the natural period of the system can be used to determine the correct design spectral ordinate and the distribution of amplification over the height of the structure.

The reference acceleration is calculated as function of the three limit states defined in BS EN 1998-3:2004, so that the demand force is:

- F_{DNC}: near collapse. Calculated for a seismic action with a probability of exceedance of 2% in 50 years.
- F_{DSD}: significant damage. Calculated for a seismic action with probability of exceedance of 10% in 50 years;
- F_{DDL}: damage limitation. Calculated for a seismic action with probability of exceedance of 20% in 50 years;

All the sub-components of the strength-only portion of the anchor assembly are brittle, or, in the case of the grouted steel elements, are not supposed to experience large deformations; therefore, they are dimensioned in terms of strength for Near Collapse limit state, according to BS EN 1998-3:2004. The minimum capacity in the assembly must be:

$$\min(F_{steel}, F_{a/b,bond}, F_{b/p,bond}, F_{masonry}) > F_{DU} \quad (2)$$

Capacities are calculated according the table below. It is recommended that tests are performed to determine the value of each performance parameter reaching a sufficient level of confidence.

For expediency, the table does not include the connections anchor/dissipative devices and the stops of the friction device; however, these component should also be dimensioned to resist F_{DNC}, as also required by EN 15129:2009, which states that anti-seismic devices should reach the ultimate limit state with damage, but no failure.

The dissipative elements of the devices, either hysteretic or frictional, are instead designed to be activated at the threshold of damage limitation, when cracks start opening and allow for the dissipative elements to become active. When the hysteretic devices enter the plastic field, or the friction devices start sliding, the connection between wall panels is still ensured, but the pull-out of the head of the anchorage is prevented and drift controlled.

Hence, from the point of view of force design, depending on which dissipative device is used, it must be:

$$F_{yield} = F_{DD} \text{ or } F_{//} = F_{DD} \quad (3)$$

The dissipative devices should also comply with requirements for interstorey drift of buildings undergoing seismic action. The chosen value of maximum allowable drift, d_r=0.003, for damage limitation is taken from OPCM 2005 that, on the contrary of BS EN 1998:2004, specifically refers to ordinary masonry buildings when stating drift limits. This limit is also in line with the expected drift stated in FEMA 356 (2000) for unreinforced masonry buildings at the limit state of Immediate Occupancy. The drift is decreased by the reduction factor v=0.4, which is taken from BS EN 1998:2004 and accounts for the fact that devices are designed to be used in heritage structures, which fall in the importance category III of Eurocode 8 (BS EN 1998:2004).

It is therefore:

$$d_r = \Delta_e = 0.5mm < 0.003h / v = 0.003(3000mm) / 0.4 = 22.5mm \quad (4)$$

where:

- Δ_e: the elongation of the device before yielding, in case of the hysteretic device, or before activation of the friction mechanism, in case of the frictional device;
- h: interstorey height, or vertical distance of installation of anchors. A standard distance of 3 meters has been assumed in the calculations, but anchors might need to be spaced more closely on the height of the wall to prevent substratum failures.

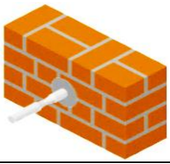
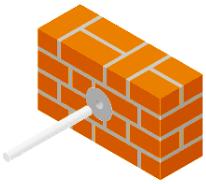
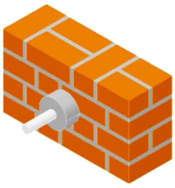
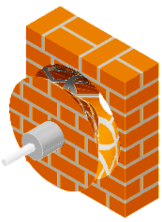
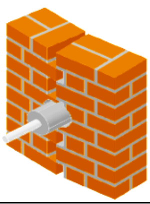
The value 0.5 mm is the first threshold of the yielding device, identified at 0.5% elongation of the dissipative element, which coincides with the 46% of maximum load of the yielding device. The second threshold identified for this type of device, 5% elongation and 72% maximum load, should instead be used to verify the capacity of the dissipative device for the limit state of significant damage, so that dissipation of energy is ensured during low-to-medium seismic excitations. Beyond this limit, the device has a further margin of safety given by buckling, meaning that the device can reach the limit of near collapse with damage, but not complete failure, and it could still be substituted, as it has been proved by experimental campaign and in response to the requirements of EN 1998:2004.

In the case of a frictional device, the drift limit is ensured by default because: the assembly is such that before activation of the friction mechanism, deformations are negligible and, beyond activation of sliding, the device displacement is limited by the assembly stops. The device can therefore perform for all limit states, as long as the connections and stops in the assembly are designed to resist up to the state of near collapse.

Further information regarding the performance thresholds of the dissipative devices can be found in D'Ayala and Paganoni (2012).

Performance parameters	Achievable range	Expected range
1a) $\mathbf{F_{yield}}$: yielding capacity of hysteretic dissipative device [kN]	$\mathbf{F_{yield}}$=33 kN (for hysteretic device of size suitable to coupling with M16 threaded bar)	$\mathbf{F_{yield}}$=27.8 kN; calculated as: $\mathbf{F_{yield}=f_{y,yield}A_{yield}}$ with $\mathbf{f_{y,yielding}}$ yielding strength of steel of hysteretic element and $\mathbf{A_{yield}}$ net cross sectional area of hysteretic element (EN 1993-1-1:2005)

1b) $\mathbf{F_{//}}$: slip-load of frictional dissipative device [kN]	Considering:
	✓ F⊥: initial value imposed on devices. Variations recorded during tests are not considered;
	✓ Slip load is given as range of values between maximum and minimum recorded values at constant level of F⊥.
	F⊥ [kN] / **$F_{//}$ min [kN]** / **$F_{//}$ Max [kN]**
	12.5 / 3.25 / 14.5
	15 / 5.7 / 18.3
	17.5 / 6.65 / 22.4
		Calculated as:
		$\mathbf{F_{//}=\Phi n F_{\perp}}$
		with Φ coefficient expressing the ratio between F⊥ and $F_{//}$, n=2 number of frictional surfaces and F⊥ applied perpendicular pressure.
		F⊥ [kN] / **$F_{//}$ (Φ=0.15)** / **$F_{//}$ (Φ=0.55)**
		12.5 / 3.75 / 13.75
		15 / 4.5 / 16.5
		17.5 / 5.25 / 19.25

2) $\mathbf{F_{steel}}$: tensile capacity of metallic bar at yielding [kN] 	$\mathbf{F_{steel}}$=71 kN (for M16 threaded bar - values stated by producer)	$\mathbf{F_{steel}}$=71 kN; calculated as: $\mathbf{F_{steel}=f_yA}$ with $\mathbf{f_y}$ yielding strength of steel and **A** net cross sectional area of metallic profile (EN 1993-1-1:2005)
3) $\mathbf{f_{b\ a/b}}$: bond strength anchor/binder [MPa] calculated on cylindrical surface of embedded bar 	Calculated as: $\mathbf{f_{b\ a/b}=F_{s/b\ bond}/A_{steel}}$ with $\mathbf{F_{s/b\ bond}}$ recorded load at failure and $\mathbf{A_{steel}}$ cylindrical lateral surface calculated as: $\mathbf{A_{steel}=\pi l d_{pitch}}$ with **l** embedment length and $\mathbf{d_{pitch}}$ pitch diameter of steel bar. For pull-out tests of M16 threaded bars from 550 mm long grouted socks: $\mathbf{f_{b\ a/b}}$=2.07 MPa (CoV 4%)	$\mathbf{f_{b\ a/b}}$= 3.4 MPa – design value suggested in BS 5268-2 for tested binder, bar diameter and type of bar 2 MPa – design value suggested in EN 1996-1-1:2005 for tested binder and type of application
4) $\mathbf{f_{b\ b/p}}$: bond strength binder/parent material [MPa] calculated on cylindrical surface of grouted socket 	Calculated as: $\mathbf{f_{b\ b/p}=F_{b/p\ bond}/A_{hole}}$ with $\mathbf{F_{b/p\ bond}}$ recorded load at failure and $\mathbf{A_{hole}}$ inner cylindrical surface of drilled hole of length **l**. For pull-out tests with vertical load on masonry specimens $\boldsymbol{\sigma_d}$: **l** [mm] / $\boldsymbol{\sigma_d}$ [MPa] / $\mathbf{f_{b\ b/p}}$ [MPa]	Calculated as: $\mathbf{f_{b\ b/p}=f_{vk}=f_{vk,0}+0.4\sigma_d}$ with $\mathbf{f_{vk,0}}$ initial shear strength (calculated through experimental results) and $\boldsymbol{\sigma_d}$ vertical load (EN 1996-1-1:2005). $\boldsymbol{\sigma_d}$ [MPa] / $\mathbf{f_{b\ b/p}}$ [MPa]
Brick masonry, f_c=6.7 MPa, f_w=0.7 MPa	350 / 0.70 / 0.67 (CoV 8%) 350 / 0.07 / 0.57 (CoV 18%)	0.7 / 0.52 0.07 / 0.27
Brick masonry f_c=3.1 MPa, f_w=0.33 MPa	220 / 0.10 / 0.26 (CoV 34%) 220 / 0.05 / 0.4	0.10 / 0.08 0.05 / 0.06
5) $\mathbf{f_{masonry}}$: Shear strength of parent material [N/mm^2] 	This type failure, although expected, did not occur during experimental campaigns	Calculated as: $\mathbf{f_{masonry}=f_{vk}=f_{vk,0}+0.4\sigma_d}$ (EN 1996-1-1:2005). In the tested case it would be expected: 0.52 MPa 0.27 MPa The failure surface, $\mathbf{A_f}$, is a truncated cone with smallest base corresponding to the drilled hole, apothem inclined at 45° and height equal to the wall thickness
6) $\mathbf{f_{masonry}}$: Tensile strength of parent material [N/mm^2] 	A “wrench” failure occurs instead of the expected “cone pull-out” failure. Failure surface, $\mathbf{A_f}$, develops along vertical joints. $\mathbf{f_{masonry}}$=f_w=0.67 MPa (from wrench test)	No mention about this type of failure has been found in the technical literature or design codes.

Execution

As the system relies on a combination of the dissipative anchoring devices with a grouted metallic anchor encased in a fabric sleeve, the execution of the intervention is similar to that of metallic anchors as described in D10.2. For sake of completeness, the procedure is reported below; for more specific indications the reader is invited to refer to D10.2, section 4.1.

Drilling

- Carefully set out the anchor position using a wax crayon or chalk, as per specifications, or as directed by the structural engineer or supervisor.
- Select the drilling method specified; for heritage buildings, due to the weakness and preciousness of the parent material, dry/ wet diamond rotary drilling rather than percussive drilling is recommended.
- Drill the hole to the required embedment depth. Remove all cores from the bore hole and check the depth. Remove dust and debris from the wall and clean all stains immediately.

Anchor Insertion and grouting

- Carefully unpack the anchor and check there has been no damage to the fabric sock during transit. Small tears or rips in the sock can be repaired using a needle and strong cotton or a hot melt glue stick. Do not shorten the length of the sock on the anchor.
- Immediately prior to insertion completely wet the sections of anchor encased in the sleeve with clean water, and position the sock evenly along the length of the anchor.
- Join the sections of the anchor, namely the standard threaded bars encased in the fabric sleeves and the dissipative device/s, which will not be embedded in grout, but will remain in the free space between one sock and the other within the borehole.
- Place the anchor in the bore hole and carefully push the anchor in, lifting it over any fissures or voids, do not force or twist the anchor into the hole.
- Install the anchor to within 50mm of the face of the brickwork (do not push completely in.). Ensure that the dissipative device is placed in correspondence of the existing crack that should be repaired, or in correspondence of the point of the connection where cracking is most likely to occur.
- When inserting the anchor ensure that the injection tubes are towards the top of the borehole - never force or twist the anchor into the hole.
- When mixing the grout, the water content can be increased by up to 10% e.g., 25 kg (56 lbs) of grout to 5.5 litres of water+ 10% = 6.05 litres. Cut mastic nozzle to fit over the injection tube.
- Proceed to inflate the sleeves from the backmost towards the front.
- When inflating anchor slowly rotate the anchor in the borehole to facilitate the grout flow and to ensure the anchor is centralised on completion. Maintain the pressure until all the grout milk has been expelled. Proceed in a similar fashion until all the sleeves in the assembly are fully grouted.

In-situ testing

Pull-out tests can be performed on-site to check the capacity of anchors according to the guidelines of D10.2.

Advantages and limits

Advantages:

- Restoration of box-like behaviour with negligible increase in mass. Horizontal loads are distributed among bearing walls according to their stiffness. Increase of mass, which is associated to the use of systems such as concrete ring beam, is avoided, thus preventing damage and collapse associated to a concentration of mass at the level of horizontal structures.
- No aesthetic impact. Anchors are embedded within the masonry and, as such they can be conceived;
- Reversibility. Anchors can be over drilled; however, this process might results intrusive and disruptive, therefore it should be avoided as much as possible;
- Brittle failures, such as punching and pull-out, which are typical of standard anchors are prevented thanks to the presence of the dissipative device
- Hysteretic device:
    - Prevention of damage in the substratum as deformation remains concentrated in the dissipative element; the size of the element needs to be carefully chosen so that yielding of the device occurs before any damage in the substratum or anchor assembly;
    - The device provides different types of performance depending on the level of acting load: elastic, frictional, elastic and plastic up to failure; therefore the system can be tailored according to design limit states;
    - Ease of maintenance: connections of the device to the anchor are designed to remain in the elastic field, so that the dissipative element can be substituted after a major earthquake.
- Frictional device:
    - Prevention of damage in the substratum as relative displacements occur by sliding of the frictional elements; the perpendicular pressure controlling the device needs to be carefully chosen and set so as to make sure that slip occurs for a tensile load lower than that causing damage in the anchor assembly or parent material.
    - Reduction of load transmitted to parent material thanks to friction load.
    - The device provides different types of performance depending on the level of acting load: elastic, frictional, elastic and plastic up to failure; therefore the system can be tailored according to design limit states;
    - Ease of maintenance: connections of the device to the anchor are designed to remain in the elastic field, so that the dissipative element can be substituted after a major earthquake.
    - Flexibility: the perpendicular pressure that controls the system performance can be tuned to suit different needs in terms of activation load, thus matching the mechanical properties of different historic substrata.

Limits

- Presence of precious finishes and geometry of the building might restrict or prevent the possibility of drilling anchors in the required position.
- Hysteretic device:
    - If the dissipative element is oversized, the dissipative anchoring device will behave as a standard anchors, i.e. pull-out or punching might still occur;
- Frictional device:
    - N/A.

Recommendations and references

D'Ayala D., Paganoni S. (2012). Protocol for Testing and Design of Dissipative Anchoring Devices. In print for Structures and Buildings (Proceeding of the ICE), Special Issue on "Strengthening of Structures under Seismic Load".

EN 1998: 2004: Eurocode 8 - Design of Structure for Earthquake Resistance.

EN 15129: 2009: Anti-seismic devices.

FEMA 356 (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings. ASCE, Washington, USA.

Paganoni S., D'Ayala D. (2009). Development and Testing of Dissipative Anchor Devices for the Seismic Protection of Heritage Buildings. ANCER Workshop 2009, Urbana-Champaign, ILL, USA, Aug 2009.

Paganoni S., D'Ayala D. (2010a). Experimental and Computational Validation of Dissipative Prototypes for the Seismic Protection of Heritage Buildings. Structural Analysis of Historic Constructions - SAHC 2010, Shanghai, China.

Paganoni S., D'Ayala D. (2010b). Dissipative Device for the Protection of Heritage Structures: a Comparison of Dynamic Tests and FE Models. XIV European Conference of Earthquake Engineering, Ohrid, Republic of Macedonia.

Paganoni S., D'Ayala D., James P. (2010). Experimental Procedure for the Validation of a Dissipative Anchor Device. Structural Faults and Repair, Edinburgh, UK.

3.1.1 Monitoring system

The monitoring anchor system is conceived as a further development of the dissipative anchoring devices and can be installed in a structure with the double function of repairing/strengthening and monitoring the response of both building and anchor. The purpose is to have a continuous monitoring of the system over either a short period immediately after a major earthquake and before the design of the permanent repairs, or in the long term as control of the performance of the strengthen structure and as an early warning system for future strong motion shaking occurrences. Hence, the system mainly aims at identifying the evolution of damage to the portion of structure where the anchor is installed by correlating the performance of the anchor itself to the response of the structure to micro-tremors, relative settlements in the ground, and so on. For instance, if the monitoring anchor is installed to restore the corner connection between two orthogonal walls, the axial deformation of the anchorage and its evolution in time can provide information on the out-of-plane behaviour of the wall.

To achieve this goal, the system needs to include a number of sensors to record relative movements – vertical and horizontal, out-of-plane and in-plane for the connected elements – that are typical of common damage mechanisms – e.g. out-of-plane overturning. Therefore, a set of sensors measuring strain/relative displacements, acceleration, and other environmental parameters are positioned at various points along the length of the anchor.

The monitoring anchor can be installed in brickwork and stonework masonry, at the connection between perpendicular walls, or walls and horizontal structures, such as timber floors in the same way standard anchor can.

The concept underlying the design of the monitoring anchor system is that it can be either used to provide information on the global structural response by positioning a number of devices at different locations within a structure, or to provide information on a specific connection by installing a single anchor in position with a higher vulnerability or interest in respect to the rest of the building. Furthermore the monitoring anchor system can be calibrated to work as early warning system: the software controlling the data acquisition can be programmed to compare recorded value to a range of admissible drifts/accelerations, so as to provide indication of dangerous evolutions of damage.

Definition and purpose

With the definition “monitoring anchor” it is herein meant a hysteretic dissipative device instrumented in such a way that allows:

- recording the deformations and accelerations in the anchor; stress field, elongation, relative displacements and other variables can be derived from the measured values. These parameters indicate the achieved level of performance of the anchor, thus contributing to the process of refinement and validation of the dissipative anchoring devices;
- recording the evolution of existing damage to a portion/subassembly of a structure – e.g. opening of a crack between two walls – and correlating it with other phenomena, such as micro-tremors on the basis of recorded deformations and accelerations. To achieve this goal the anchor system needs installing in a structure with another independent monitoring system so as to correlate the results.

The system is made of:

- one stainless steel anchor made of three sections: the first and last sections are made of standard threaded profiles, while the mid-section is made of a dissipative anchoring device. The last section of the assembly is grouted, so as to provide anchorage within the parent material, the central section is positioned in correspondence of a crack, or at the point of the connection where monitoring is required, and the front section is dry-installed, with a bolted end-plate to ensure connection and ease of removal. Various strain gauges are bonded along the anchor, with cables for the connection to the acquisition system laid along the drilled hole;

- a set of electrical resistance strain gauges. Strain gauges are bonded and coated with suitable products depending on the conditions and procedures of installation – e.g. indoor/outdoor installation. Strain gauges are selected from the wide range of commercially available gauges in the light of each specific installation;
- strain gauge amplifiers. Each bridge will require a separate amplifier;
- a tri-axial accelerometer;
- a temperature sensor fitted near the anchor;
- embedded computer running Microsoft Windows XP (FES). This gives a platform for the control of data acquisition, processing and storage of data and for supporting remote communications;
- data acquisition card;
- enclosure and assembly. All of the hardware is securely contained in one enclosure. There are various ports on the outside of the enclosure for connections to be made. There is a fused power inlet socket for a 220-240 V ac supply;
- bespoken software. A continuously running program causes the data acquisition card to sample all channels at a set rate. At the end of a specified period, in this case 1 hour, the acquired data is written to a binary block on the hard disk using a name that includes the date and time at which the block started. Summary statistics for each channel such as maximum, minimum, mean and root mean square (RMS) are also output to an ASCII file suitable for viewing with Excel.

Other sensors can be added depending on the specific application.

Applicability conditions

The system needs continuous electrical supply; it is advised that a UPS is put in series with the system so as to ensure that in case of loss of power the system is able to carry on recording data for at least a couple of hours.

For remotely controlling the system, continuous internet connection is needed; this can be either through the phone line or through the mobile network.

The data logger needs to be positioned in a dry, safe indoor environment.

Cables need to be run from the anchor to the data logger; it is therefore suggested that the data acquisition system is positioned as closed as possible to the anchor, so as to avoid that the length of the cables might hinder the quality and speed of the data acquisition. In case of building where damage is already present, it might be possible to run the cable within the crack; otherwise a small recess needs to be cut into the parent material to run the cables to the data logger. It is recommended that cable remain within the building, and are repaired by environmental agents.

In regards to the anchor in general the same conditions as per paragraph “Applicability Conditions” in section 3.1 apply.

Design

The following factors should be considered in order to decide what instruments are to be embedded in the anchor system:

- a set of electrical resistance strain gauges is embedded in the system. Depending on the damage mechanism that are expected to occur and need to be monitored, the number and the type of strain gauges will change. Considering that the anchor will be used to connect a masonry wall to an orthogonal wall or to a horizontal structure, the following relative motions can be monitored:
    - out-of-plane motion of front wall, recorded by axial and vertical bending bridges;

    - out-of-plane motion of side wall/horizontal structure, detected by horizontal shear and bending bridges;
    - vertical relative displacements between the two walls/wall and horizontal structure, captured by vertical shear and bending bridges;
    - in-plane horizontal movements of the front wall, detected by shear and bending bridges.
- the number of strain gauge amplifiers included in the monitoring system will be decided on the basis of the number of strain gauge bridges;
- the data acquisition card need to be chosen depending on the number and typology of used sensors.

Execution

1. Carry out drilling according to the recommendation listed in section 3.1, “Execution”, “Drilling”.
2. Carefully unpack the anchor and check there has been no damage to the fabric sock or to the instrumentation cabling during transit.
3. Immediately prior to insertion wet completely the sections of anchor encased in the sleeves with clean water, and position the sock evenly along the length of the anchor.
4. Join the sections of the anchor, namely the standard threaded bars encased in the fabric sleeves and the dissipative instrumented device/s, which will not be embedded in grout, but will remain in the free space between one sock and the other within the borehole.
5. Place the anchor in the bore hole and carefully push the anchor and the cable connecting the instrumentation to the data logger in, lifting it over any fissures or voids; do not force or twist the anchor into the hole.
6. Install the anchor to within 50mm of the face of the brickwork (do not push completely in.). Ensure that the dissipative device is placed in correspondence of the existing crack that should be repaired, or in correspondence of the point of interest for the monitoring. Run the cables within the crack into the building to the position where the data logger will be positioned. If no cracking exists at the moment of the installation, a recess needs cutting in the masonry so as to be able to bring the cables to the data logger.
7. When inserting the anchor ensure that the injection tubes are towards the top of the borehole - never force or twist the anchor into the hole.
8. When mixing the grout, the water content can be increased by up to 10% e.g., 25 kg (56 lbs) of grout to 5.5 litres of water+ 10% = 6.05 litres. Cut mastic nozzle to fit over the injection tube.
9. Proceed to inflate the sleeves proceeding from the backmost one to the front one.
10. When inflating anchor slowly rotate the anchor in the borehole to facilitate the grout flow and to ensure the anchor is centralised on completion. Maintain the pressure until all the grout milk has been expelled. Proceed in a similar fashion until all the sleeves in the assembly are fully grouted.
11. Position and lock the end plate with three-axial accelerometer at front part of the anchor;
12. Wire the cables to the data acquisition system and UPS.

In-situ testing

Pull-out tests can be performed on-site to check the capacity of anchors according to the guidelines of D10.2.

Simple checks can be performed on the instrumentation prior to installation and grouting in order to ensure that it is working correctly. This is done by wiring the cables to the data logger, switching on the system and:

- applying a tension or slightly bending the bars, so as to check whether a variation in strains is recorded by the system;
- quickly moving the accelerometer in three orthogonal directions, so as to ensure that the variation in acceleration is read by the system;
- slightly warming up the temperature gauge to check that a variation of temperature is recorded by the system. In cold environments, body heat is sufficient to provoke a variation in recorded temperature, so hold the gauge in the hands until an increase in temperature is recorded.

Advantages and limits

Advantages:

- a joint strengthening/monitoring system whereby two different tasks for the protection of heritage assets, which normally involves the use of separate systems, can be carried out together;
- a complex, yet compact monitoring device that can read the possible relative movements between two macro-elements, allowing for the characterisation of the structural behaviour of the connection, but also of the whole structure when more than one anchor is installed in the building. Whereas a standard monitoring system detects the response of a structure at a global level, the instrumented anchor can capture localised phenomena, such as the load experienced by the strengthening element or the stress transmitted to the parent material. The system is designed and assembled in house so that installation of a number of separate sensors is avoided, reducing the amount of resources needed and the risks for contractors working on site. Still the information output is equivalent or higher than for a standard monitoring system;
- once the system is fully calibrated, a software for the analysis of the recorded data and the comparison with a set of limit values is developed, so that in the case of urgent interventions, when data are required in a short time, the whole process of monitoring is simplified: critical locations are identified, the monitoring anchors installed and monitoring is performed with no need for further calibration.

Limits:

- as other monitoring and early warning systems is affected by availability of power supply and velocity of data transmission and processing in respect to the velocity of the phenomenon affecting the structure.

3.2 STICK-AND-SLIP CARPENTRY CONNECTIONS

Stick and slip dovetail joints		
Design parameters	*Applicability*	*Advantages and limts*
- Ductility of rods (bolt) - Friction of break plates - Friction of oak plates	Real joints of historic structures with high deformation during horizontal wind or earthquake actions	Advantages: - minimum interventions; - simple design. - solutions do not require total disassembly of the structure. Limits: - The effect is dependent on the force pressing the both surfaces together.

Definition and purpose

The purpose of cyclic tests is to analyse the effectiveness of strengthening interventions on dovetail halved joints from the point of view of dissipative properties and change in stiffness. Different typologies of strengthening, like the addition of combined damping/reinforcing elements, e.g. steel nails, or damping elements only, e.g. brake plates, are investigated in order to describe the influence of various parameters on the improvement of the seismic capacity of the joint.

The retrofitting approach is constrained by the conservation requirement for minimum interventions. Several retrofitting methods have been adopted and tested on replicas of historic halved joints made of ancient material taken from a demolished building using traditional carpenter's tools and processes.

In the proposed case, the energy dissipation capacity is increased by inserting two thin plates in the joint and by substituting the wooden pin with a bolt. During the motion of the timber structure, the connected timber struts rotate mutually in the joint and tend to deform the inserted steel bolt plastically, which absorbs the energy. The plates are made of a material with a high friction coefficient. Disc brake plates or even thin oak plates can be used.

Applicability conditions

For a typical baroque and a typical gothic roof framework in which halved dovetail joints have been widely applied. The results have approved that well done tight joints with reduced slip possibilities decrease the overall roof frame deformation during horizontal wind or earthquake loads, together with a reasonably high rotational stiffness and capacity. Highly skilled carpenters were able to make perfect joints without gaps between individual elements.

In the case of historic structures, the retrofitting approach is constrained by the conservation requirement for minimum interventions. The interventions should not severely change the behaviour and appearance of the structure, and solutions that do not require total disassembly of the structure should be preferred.

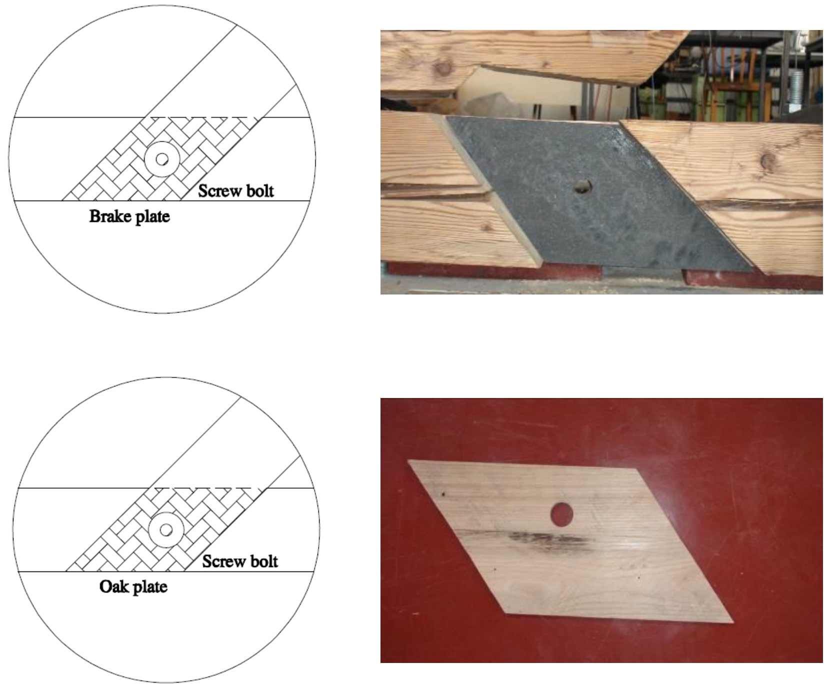
Design

The main design parameters for high friction coefficient plates and oak plates, considered for strengthening intervention are:

Typology of strengthening	Performance parameters	Achievable range	Expected range
Combination of: a) Glued carbon fibre plates b) Brake plates c) Oak plates and strengthening elements oblique to plane of joint. (nails, woodscrews, bolts)	- Friction between materials used for the strengthening (-)	- Depending on type of plate, on fixing to timber elements and on applied prestress.	○ Coefficient of friction (oak – oak) - μ = 0.4 ○ Coefficient of friction (brake plate – brake plate) - μ = 0.4 ○ Coefficient of friction (spruce – spruce) - μ = 0.28 (Leonardo da Vinci P.P, 2008; http://www.sittech.cz/brzdove/6800.htm)
	- Prestress (Nm)	- During tests value of 90 and 240Nm are used	- Depending on deformation of wood (2.0 -2.5N/mm^2) (Leonardo da Vinci P.P, 2008)
	- Load for activation of frictional mechanism (N)	- Dissipated energy expressed as percentage of improvement in respect to unreinforced joint: ○ Carbon fibres: 100% ○ Brake plates, depending on prestress: - 90 Nm 150% - 115 Nm 215% - 240 Nm 305% ○ Oak plates, depending on prestress: - 90 Nm 120% - 115 Nm 180% - 170 Nm 250%	- The values of load of activation of frictional mechanism were determined only by test. Dissipated energy expressed as percentage of improvement in respect to unreinforced joint: ○ Carbon fibres: 100% ○ Brake plates, depending on prestress: - 90 Nm 80% - 115 Nm 120% - 240 Nm 200% ○ Oak plates, depending on prestress: - 90 Nm 50% - 115 Nm 80% - 170 Nm 125%

Execution

The connecting wooden pin is removed from the joint during intervention, the halved parts are slightly opened, and two thin plates are inserted in the opened slot and are stuck to the wooden elements. The plates are made of a material with a high friction coefficient. Disc brake plates and thin oak plates were used. The joint is then fixed and tighten with a steel bolt which is prestressed to a certain level. The screw bolt allows changing a level of the prestress of the joint optionally which influences not only a stiffness of the joint but also a friction force between the plates. Several prestress values were chosen and tested (from 115 Nm up to 240 Nm - which generated stress level on the friction surfaces of the order of about 0.43 MPa to 0.9 MPa.)



Replacement of the oak pin by a steel screw bolt.

Two brake plates freely inserted between contact surfaces of the beams.

Replacement of the oak pin by a steel screw bolt.

Two oak plates freely inserted between contact surfaces of the beams

Figure 3.1 - Principle of joint retrofitting with friction plates.

Advantages and limits

Advantages:
- minimum interventions;
- simple design;
- solutions do not require total disassembly of the structure.

Limits:
- the effect is dependent on the force pressing the both surfaces together;
- maximum value of the friction force is limited due to the compressive deformation of wood.

Recommendations and references

Drdácký M., Wald F., Sokol Z. (1999). Sensitivity of Historic Timber Structures to Joint Response. XC Anniversary Congress of IASS Madrid (ed. R. Astudillo and A. J. Madrid), Vol. II Madrid, CEDEX Madrid, G1-G10.

Drdácký M., Wald F., Mareš J., Sokol Z. (2000). Component method for historical timber joints. The Paramount Role of Joints into the Reliable Response of Structures (ed. C. C. Baniotopoulos and F. Wald), NATO Science Series, ISBN 0-7923-6701-4 (PB), ISBN 0-7923-6700-6 (HB) Dordrecht/Boston/London, Kluwer Academic Publishers, pp. 417-424.

Kasal B., Pospíšil S., Jirovský I., Heiduschke A., Drdácký M., Haller P. (2004). Seismic performance of laminated timber frames with fiber-reinforced joints. Journal of Earthquake Engineering and Structural Dynamics, No. 33, pp. 633-646.

3.3 DUCTILE ANCHORS

DUCTILE ANCHORS		
Design parameters	*Applicability*	*Advantages and limts*
- Yielding strength of ductile anchor plate; - Tensile strength of steel tie rod; - Compressive strength of parent material; - Shear strength of parent material; Diameter of tie rod.	Generally applicable. In case of heritage buildings, the anchor plates can be concealed	*Advantages* - Minor aesthetic impact; - Reversibility; - Replaceable ductile plate - Prevention of damage in historic substrate - Ductile failure; - Box-like behaviour of the structure. *Limits* - Architectural restrictions might narrow the application of the ductile plates; If oversized, the ductility of the plates isn't explored. Failure modes related to the wall and tie rod will occur.

Definition and purpose

The ductile anchor plates were developed by Monumenta and University of Minho, to ensure a global behaviour of masonry buildings and avoid local out-of-plane mechanisms of walls. These enhanced anchor plates take advantage of their shape to increase ductility and have a better performance, under seismic action, than the standard (flat) anchor plates. The ductile plates are circular and have a double curved shape with a uniform thickness of 6 mm. The external diameter is 250 mm while the internal one is 65 mm. The six notches are 8 mm wide and axisymmetrically distributed every 60°. The plate itself is curved, having only two horizontal surfaces, at the top where it rests the hinge (half spherical cup) to which the tie rod is anchored and at the contact surface with the substrate. To better distribute the seismic action, the solution contemplates a complementary base plate between the ductile anchor plate and the substrata. The solution is intended for wall-to-wall connections but can also improve wall-to-floor connections, when applied at floor level.

This systems aims at restoration of box-like behaviour and prevention of out-of plane mechanisms of masonry walls. If correctly designed, it will reduce the risk of damage to the wall during seismic action, meaning that failure modes like formation of the shear pull-out cone (punching) and crushing of the masonry under the anchor plate will be prevented. Consequently, costs of repair will be reduced. To sum up, the ductile anchor plates aim at increased ductility of the connection, more energy dissipation and less damage on the structure.

Applicability conditions

The strengthening solution has general application on masonry buildings and is expected to perform better than standard systems of anchor plates and tie rod. In heritage buildings, concerns with architectural detailing can compromise its application on the required position. This can be overcome, by studying solutions to conceal the system.

In general application and especially in case of poor parent material, it is recommended improvement of the substrata under the anchor plate. This can be achieved by a reinforced mortar cushion between the substrata and the ductile anchor plate.

Design

The determination of the design seismic forces will address unreinforced masonry buildings with flexible timber diaphragms, as described in NZSEE (2006). For the calculation of the seismic demand (F_i), Modal Analysis or the Equivalent Static Method can provide good estimations of the design values.

$$F_i \leq F_d \quad (5)$$

With F_i being the horizontal seismic force at storey i and F_d the design value of resistance. This last value corresponds to the minimum resistance associated with the possible failure modes that can occur: crushing of the masonry under the anchor plate ($F_{d,c}$), punching of the masonry ($F_{d,p}$), yielding of the tie rod ($F_{d,tr}$) and failure of the ductile anchor plate ($F_{d,dp}$). To achieve the intended behaviour, the minimum resistance should correspond to the one of the ductile anchor plate, $F_{d,dp,}$ or to the one of the tie rod, $F_{d,tr}$, meaning failure modes related to masonry will not occur. The capacity of the masonry wall, $F_{d,mas}$, is the minimum of $F_{d,c}$ and $F_{d,p}$. Since the single ductile anchor plate has constant characteristics and properties, capacity will be achieved by applying several per floor (n_t), according to EN 1996:2005: EC6. The design capacity of the ductile plate was determined according to the compressive tests carried out in UMinho and its value is 90 kN. $F_{d,c}$ and $F_{d,p}$ depend mainly on the properties of the masonry and on the geometry of the base plate, and can be calculated according to () and (), respectively. The cross-section diameter of the tie rod can be estimated with the expression shown in (). The ductile anchor plate can be prepared to anchor the following cross-section diameters, in mm, of tie rod: 16, 20, 25 e 32.

$$F_{d,c} = f_c \times A_{bp}$$
$$A_{bp} = \frac{\phi_{bp}^2 \times \pi}{4} \quad (6)$$

where f_c is the compressive strength of the masonry, A_{bp} is the area under the base plate and ϕ_{bp} is the diameter of the base plate.

$$F_{d,p} = f_{vk} \times A_l$$
$$A_l = \frac{\pi}{2}\left(\frac{d_2 \times t}{\cos 45} - \frac{d_1}{2 \sin 45}\right) \quad (7)$$

where f_{vk} is the shear strength of the masonry and Al is the area of the conical failure surface. Other dimensions used for the calculation of the area are represented in Figure 3.2.

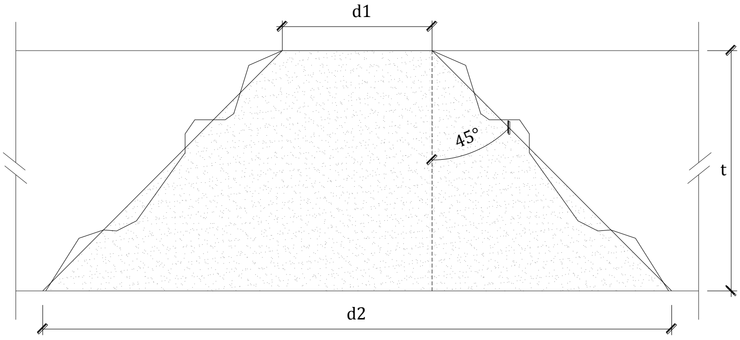
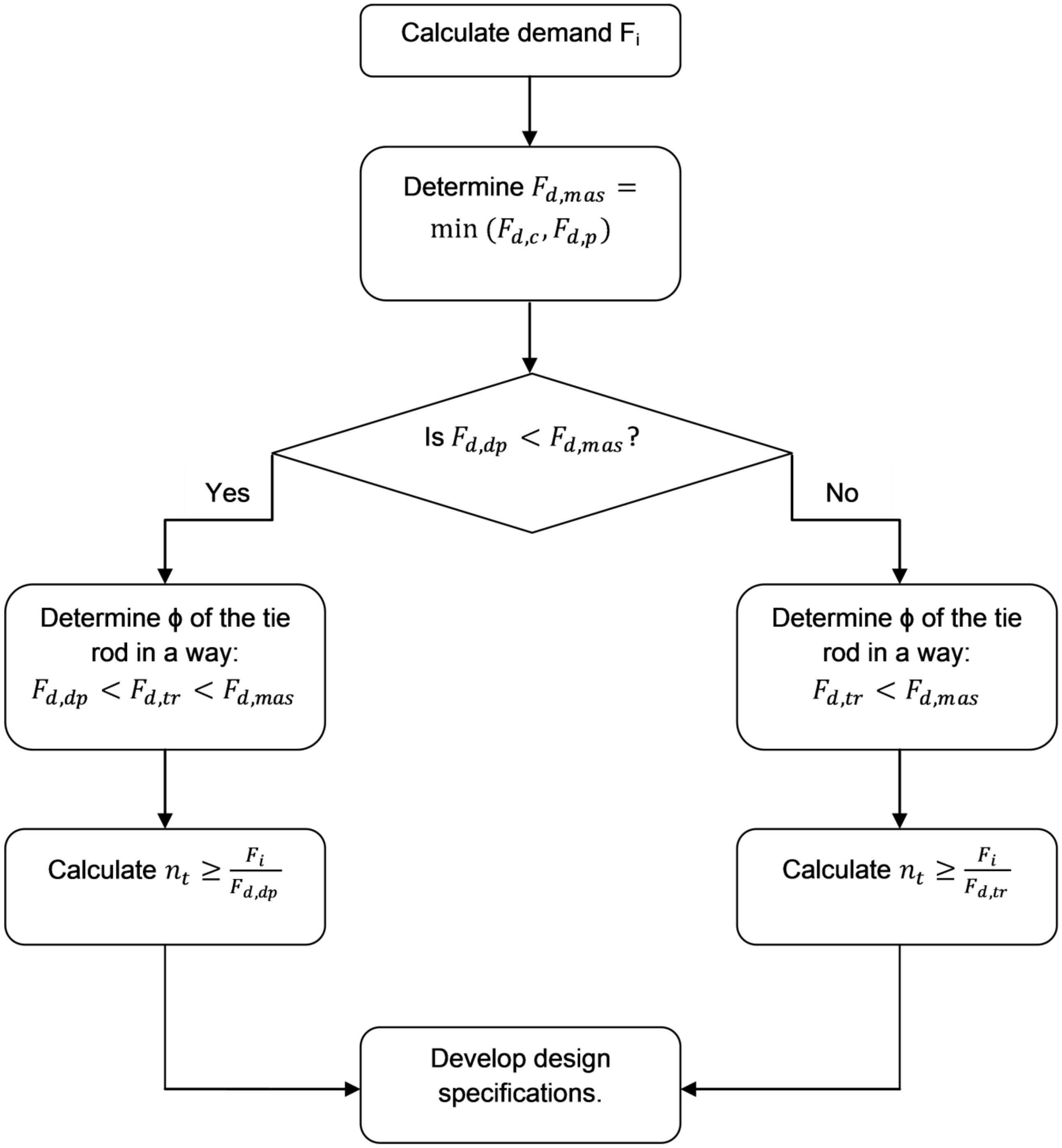


Figure 3.2 - Failure surface of punching .

$$A_{tr} = \frac{F_{d,tr}}{f_y}$$
$$\phi_{tr} = \sqrt{\frac{A_{tr} \times 4}{\pi}} \quad (8)$$

where f_y is the yield strength and A_{tr} is the cross-section area.
Considering all these variables and assumptions, design should follow the flowchart bellow.



Execution

As described previously, the strengthening system consists of two main components: the tie rod and the ductile anchor plate. In the case of the double anchor plate, besides these two components there is a distribution arm connecting both plates. To install the system, a set of tasks should be carried out: positioning the anchor plates, drilling the wall, preparation of the substrate, tie rod insertion, anchor plates placing and tie rod tensioning.

Positioning

- Carefully set out the anchor position using a wax crayon or chalk, as per specifications, or as directed by the structural engineer or supervisor.

Drilling

- Select the drilling method specified; for heritage buildings, due to the weakness and preciousness of the parent material, dry/ wet diamond rotary drilling rather than percussive drilling is recommended;
- Drill the hole throughout the entire thickness of the wall. Remove all cores from the bore hole. Remove dust and debris from the wall and clean all stains immediately.

Preparation of the substrate

- Using wax crayon or chalk, draw the boundaries of the surface where the mortar will be placed, as specified;
- Remove dust and wet the surface with clean water, to improve adhesion between mortar and substrate;
- Prepare the mix of mortar, according to specifications, and apply a first layer on the wall;
- Place the steel mesh on the mortar and apply a second layer;
- Let it cure according to project specifications;

Tie rod insertion

- Carefully unpack the tie rod and check if there has been damage to it during transit;
- Place the tie rod in the bore hole, leaving 100 mm past the external face of the wall;

Anchor plates placing

- Insert the base plate and then anchor plate trough the tie, and screw the nuts;

Tie rod tensioning

- Tighten the nuts with a torque wrench to a specified force;
- Using turnbuckles, adjust the tension of the tie rods.

In-situ testing

When possible, stone and mortar samples should be collected to study their mechanical properties.

Advantages and limits

Advantages:

- box-like behaviour ensured, by preventing out-of-plane collapses of walls;
- minor aesthetic impact. Anchors plates are visible on the external face of the wall but are relatively small when compared to the scale of the building;
- reversibility. Tie rods and anchor plates can be removed, with minimum damage to the wall, before and after seismic action;
- ductile anchor plate:

  - Appropriate design prevents failure modes related with the masonry wall, like formation of pull-out cone or crushing of masonry under the anchor plate;
  - The device is design to perform in the plastic range, up to failure, so that its dissipative capacity is fully explored;
  - Tie rod and distribution arm are designed to work in the elastic range, so that the ductile plate can be substituted after a major earthquake.

Limits:

- architectural restrictions, especially on heritage buildings, can limit its application. The preservation of architectural finishes can restrict the application of the strengthening in the required position;
- ductile anchor plate:
  - If the ductile anchor plate is oversized, larger deformations will occur in the distribution arm and tie rod, and formation of a pull-out shear cone can occur in the masonry wall.

Recommendations and references

Appleton J. (2005). Reabilitação de edifícios gaioleiros. Lisboa: Edições Orion.

Cóias V. S. (2007). Reabilitação estrutural de edifícios antigos. Lisboa: ARGUMENTUM GECoRPA.

EN 1996:2005: Eurocode 6 – Design of masonry structures.

EN 1998:2004: Eurocode 8 – Design of Structure for Earthquake Resistance.

FEMA 356 (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings. ASCE, Washington, USA.

Mascarenhas J. (2005). Sistemas de Construção V - O Edifício de Rendimento da Baixa Pombalina. Materiais Básicos: o Vidro - 2ª edição revista e actualizada. Lisboa, Livros Horizonte.

Ramos L. F., Lourenço P. B. (2004). Modelling and vulnerability of historical city centers in seismic areas: a case study in Lisbon. Engineering Structures, pp. 1295–1310.

NZSEE (2006). Assessment and improvement of the structural performance of buildings in earthquakes: Recommendations of a NZSEE study group on earthquake risk buildings. New Zealand Society for Earthquake Engineering.

Cardoso R., Bento R., Lopes M. (2005). Seismic evaluation of old masonry buildings. Part I: Method description and application to a case-study. Engineering Structures, pp. 2024-2035.

Bento R., Lopes M., Cardoso R. (2005). Seismic evaluation of old masonry buildings. Part II: Analysis of strengthening solutions for a case-study. Engineering Structures, pp. 2014-2023.

4 CONCLUSIONS

In this document the recommendations for a correct choice, installation and design of dissipative systems are reported. In the field of heritage structures, although the use of ductility and energy-based systems is provided for and encouraged by current codes, traditional stiffness-based systems are still widely applied. However ductility-based systems could instead tackle the problem of brittle failures by allowing controlled relative displacements or rotations, limiting the load transferred to the original materials and improving the dissipation of energy at the joint.

Accordingly, a set of dissipative devices have been developed and validated by the NIKER consortium’s partners. These include the following:

Dissipative anchoring devices w/o monitoring aim to restore the unitary behaviour of a structure by ensuring the connection between sets of perpendicular walls and to reduce the risk of out-of-plane mechanisms of masonry panels. Additionally, the devices also aim at preventing brittle failures at the head of the anchorage, such as punching and pull-out, which normally affect metallic anchor, both in the set-up with end plate or fully grouted; allowing relative controlled displacements between two walls, thus ensuring ductility of the connection and dissipation of energy within the standard drift limits prescribed by codes; reducing the load transmitted to the weak substratum by the anchorage. These goals are reached by means of either a stainless steel element, shaped to optimise its post-elastic behaviour, or a device relying on a friction mechanism set to be triggered for a certain level of pulling/pushing force; the devices are placed in series with a metallic grouted anchor, in correspondence of an existing crack, or where damage is most likely to occur as consequence of the poor quality of connections or simply of the wear and tear of the structure.

Monitoring anchor system is conceived as a further development of the dissipative anchoring devices and can be installed in a structure with the double function of repairing/strengthening and monitoring the response of both building and anchor. The purpose is to identify the evolution of damage to the portion of structure where the anchor is installed by correlating the performance of the anchor itself to the response of the structure to micro-tremors, relative settlements in the ground, and so on.

Stick-and-slip carpentry connections allow increasing the energy dissipation capacity by inserting two thin plates of a high friction coefficient material in the opened slot and are stuck to the wooden elements. Disc brake plates or even thin oak plates can be used. During the motion of the timber structure, the connected timber struts rotate mutually in the joint and tend to deform the inserted prestressed bolt plastically, which absorbs the energy.

Ductile anchor plates take advantage of their shape to increase ductility and have a better performance, under seismic action, than the standard (flat) anchor plates. This systems aims at restoration of box-like behaviour and prevention of out-of plane mechanisms of masonry walls. If correctly designed, it will reduce the risk of damage to the wall during seismic action, meaning that failure modes like formation of the shear pull-out cone (punching) and crushing of the masonry under the anchor plate will be prevented. Consequently, costs of repair will be reduced.

5 REFERENCES

Aiken I.D., Kelly J.M., Pall A.S. (1988). Seismic Response of a Nine-story Steel Frame with Friction Damped Cross-Bracings. IX World Conference of Earthquake Engineering.

Aiken I.D., Kelly J.M., Pall A.S. (1993). Testing of Passive Energy Dissipation Systems. Earthquake Spectra, Vol. 9, No. 3, pp. 335-370.

Appleton, J. (2005). Reabilitação de edifícios gaioleiros. Lisboa: Edições Orion.

Benedetti D. (2004). Increasing Available Ductility in Masonry Buildings via Energy Absorbers. Shaking Table Tests. European Earthquake Engineering, No. 3.

Cardoso R., Bento R., Lopes M. (2005). Seismic evaluation of old masonry buildings. Part I: Method description and application to a case-study., Engineering Structures, pp. 2024-2035.

Cardoso R., Bento R., Lopes M. (2005). Seismic evaluation of old masonry buildings. Part II: Analysis of strengthening solutions for a case-study. Engineering Structures, pp. 2014-2023.

Chang C.Y.L., Pall A., Louie J.J.C. (2006). The Use of Friction Dampers for Seismic Retrofit of the Monterey County Government Centre. US National Conference on Earthquake Engineering. Oakland, CA, USA.

Christopoulos C., Tremblay R., Kim H.J., Lacerte M. (2008). Self-Centring Energy Dissipative Bracing System for the Seismic Resistance of Structures: Development and Validation. Journal of Structural Engineering, Vol. 134, No. 1, pp. 96-107.

Cóias V.S. (2007). Reabilitação estrutural de edifícios antigos. Lisboa: ARGUMENTUM GECoRPA.

Constantinou M.C., Symans M.D. (1993) Seismic response of Structures with Supplemental Damping. The Structural Design of Tall Buildings, Vol. 2, No. 2, pp. 77-92.

D'Ayala D., Paganoni S. (2012). Protocol for Testing and Design of Dissipative Anchoring Devices. In print for Structures and Buildings (Proceeding of the ICE), Special Issue on Strengthening of Structures under Seismic Load.

Drdácký M., Wald F., Sokol Z. (1999). Sensitivity of Historic Timber Structures to Joint Response. XL Anniversary Congress of IASS Madrid, Vol. II, Madrid, CEDEX Madrid, G1-G10.

Drdácký M., Wald F., Mareš J., Sokol Z. (2000). Component method for historical timber joints. The Paramount Role of Joints into the Reliable Response of Structures (ed. C. C. Baniotopoulos and F. Wald), NATO Science Series, ISBN 0-7923-6701-4 (PB), ISBN 0-7923-6700-6 (HB) Dordrecht/Boston/London, Kluwer Academic Publishers, pp. 417-424.

EN 1996 (2005). Eurocode 6 – Design of masonry structures.

EN 1998 (1998). Eurocode 8 – Design of Structures for Earthquake Resistance.

EN 15129 (2009). Anti-seismic devices.

FEMA 356 (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings. ASCE, Washington, USA.

Indirli M., Castellano M.G. (2008). Shape Memory Alloy Device for the Structural Improvement of Masonry Heritage Structures. International Journal of Architectural Heritage, No. 2, pp. 93-119.

Kasal B., Pospíšil S., Jirovský I., Heiduschke A., Drdácký M., Haller P. (2004). Seismic performance of laminated timber frames with fiber-reinforced joints. Journal of Earthquake Engineering and Structural Dynamics, No. 33, pp. 633-646.

Lopez Garcia D., Soong T.T. (2002). Efficiency of a Simple Approach to Damper Allocation in MDOF Structures. Journal of Structural Control, Vol. 9, No. 1, pp. 19-30.

Lin W., Chopra A.K. (2002). Earthquake Response of Elastic SDF Systems with Non-linear Fluid Viscous Dampers. Earthquake Engineering and Structural Dynamics, Vol. 31, No. 9, pp. 1623-1642.

Mandara A., Mazzolani F.M. (1994). Seismic Upgrading of Churches by Means of Dissipative Devices. STESSA: Behaviour of Steel Structures in Seismic Areas.

Mascarenhas J. (2005). Sistemas de Construção V - O Edifício de Rendimento da Baixa Pombalina. Materiais Básicos: o Vidro - 2ª edição revista e actualizada. Lisboa: Livros Horizonte.

Morgen B.G., Kurama Y.C. (2008). Characterization of Two Friction Interfaces for Use in Seismic Damper Applications. Materials and Structures, Vol. 42, No. 1, pp. 35-49.

Paganoni S., D'Ayala D. (2009). Development and Testing of Dissipative Anchor Devices for the Seismic Protection of Heritage Buildings. ANCER Workshop 2009, Urbana-Champaign, ILL, USA, Aug 2009.

Paganoni S., D'Ayala D. (2010a). Experimental and Computational Validation of Dissipative Prototypes for the Seismic Protection of Heritage Buildings. Structural Analysis of Historic Constructions - SAHC, Shanghai, China.

Paganoni S., D'Ayala D. (2010b). Dissipative Device for the Protection of Heritage Structures: a Comparison of Dynamic Tests and FE Models. XIV European Conference of Earthquake Engineering, Ohrid, Republic of Macedonia.

Paganoni S., D'Ayala D., James P. (2010). Experimental Procedure for the Validation of a Dissipative Anchor Device. Structural Faults and Repair, Edinburgh, UK.

Priestley M.J.N. (2000). Performance Based Seismic Design. XII World Conference of Seismic Engineering, Auckland, New Zealand.

Ramos L.F., Lourenço, P.B. (2004). Modelling and vulnerability of historical city centers in seismic areas: a case study in Lisbon., Engineering Structures, pp. 1295–1310.

Rodgers G.W., Mander J.B., Chase J.G.,Dhakal R P., Leach N.C., Denmead C.S. (2008). Spectral Analysis and Design Approach for High Force-to-volume Extrusion Damper-based Structural Energy Dissipation. Earthquake Engineering and Structural Dynamics, Vol. 37, No. 2, pp. 207-223.

RNZSEE (2006). Assessment and improvement of the structural performance of buildings in earthquakes: Recommendations of a NZSEE study group on earthquake risk buildings, New Zealand Society for Earthquake Engineering.

Symans M.D., Charney F.A., Whittaker A.S., Constantinou M.C., Kircher C.A., Johnson M.W., McNamara R.J. (2008). Energy Dissipation Systems for Seismic Applications: Current Practice and Recent Developments. Journal of Structural Engineering, Vol. 134, No. 1, pp. 3-21.