Rehabilitation and Maintenance of Buildings

Valeriu STOIAN

Lecture : 19/06/2014 – 8.30-10.00/10.30-12.00

European Erasmus Mundus Master Course

Sustainable Constructions
under Natural Hazards and Catastrophic Events

520121-1-2011-1-CZ-ERA MUNDUS-EMMC

Universitatea Politehnica Timişoara

COMPOZITES ?

FIBER

- carbon
- glass
- aramid
- hybride

+

RESIN

- thermoplast
- thermorigide

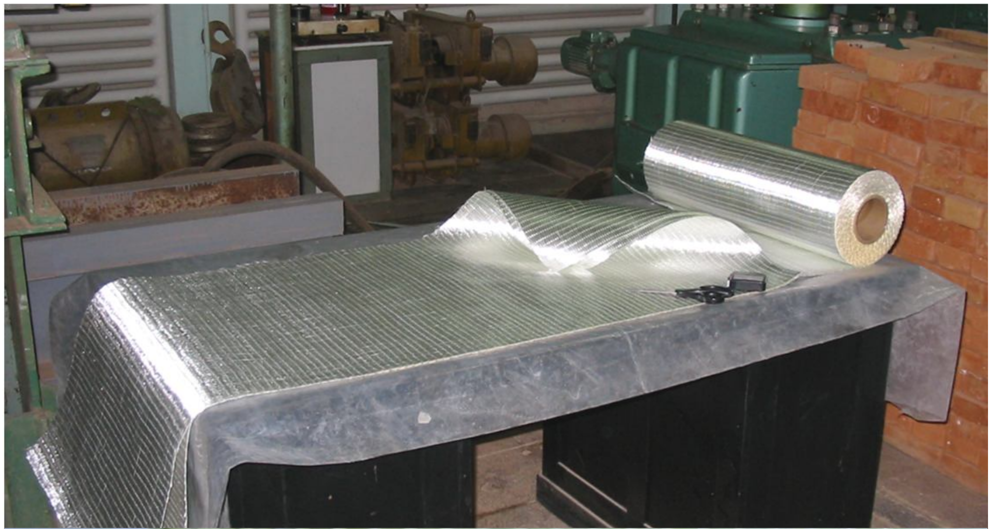
↓

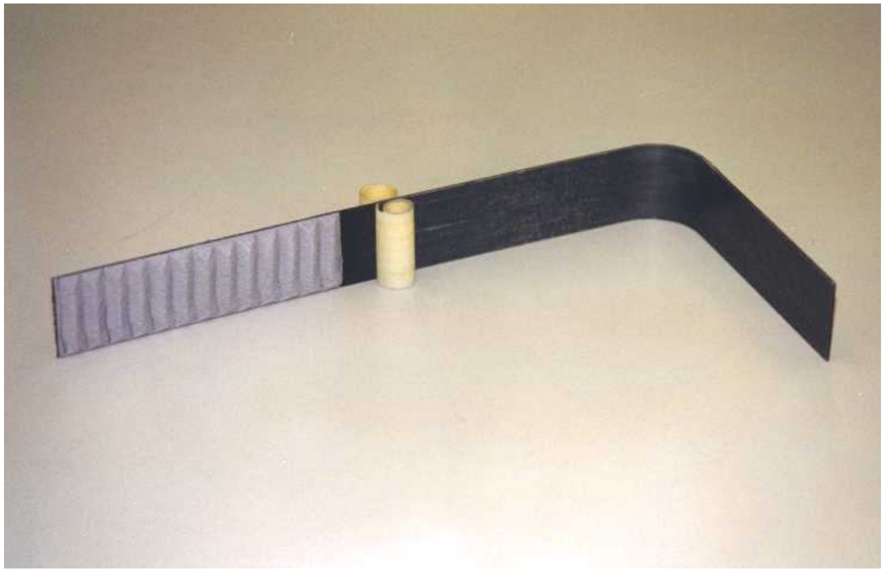
COMPOSITE (FRP)

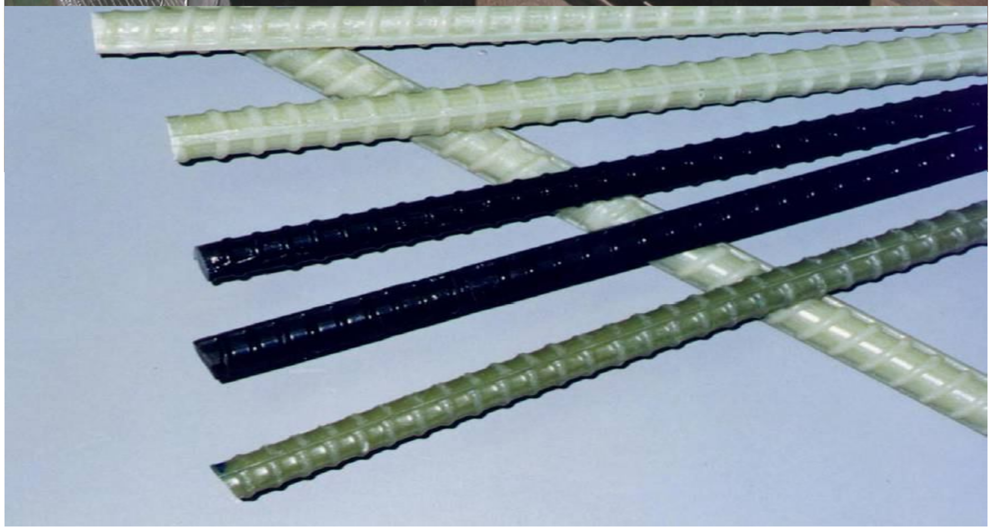
(polymers reinforced with fiber)

FRP TYPES

1. STRENGTHENING
   - FABRIC
   - LAMELAS (STRIP)
   - REINFORCEMENT

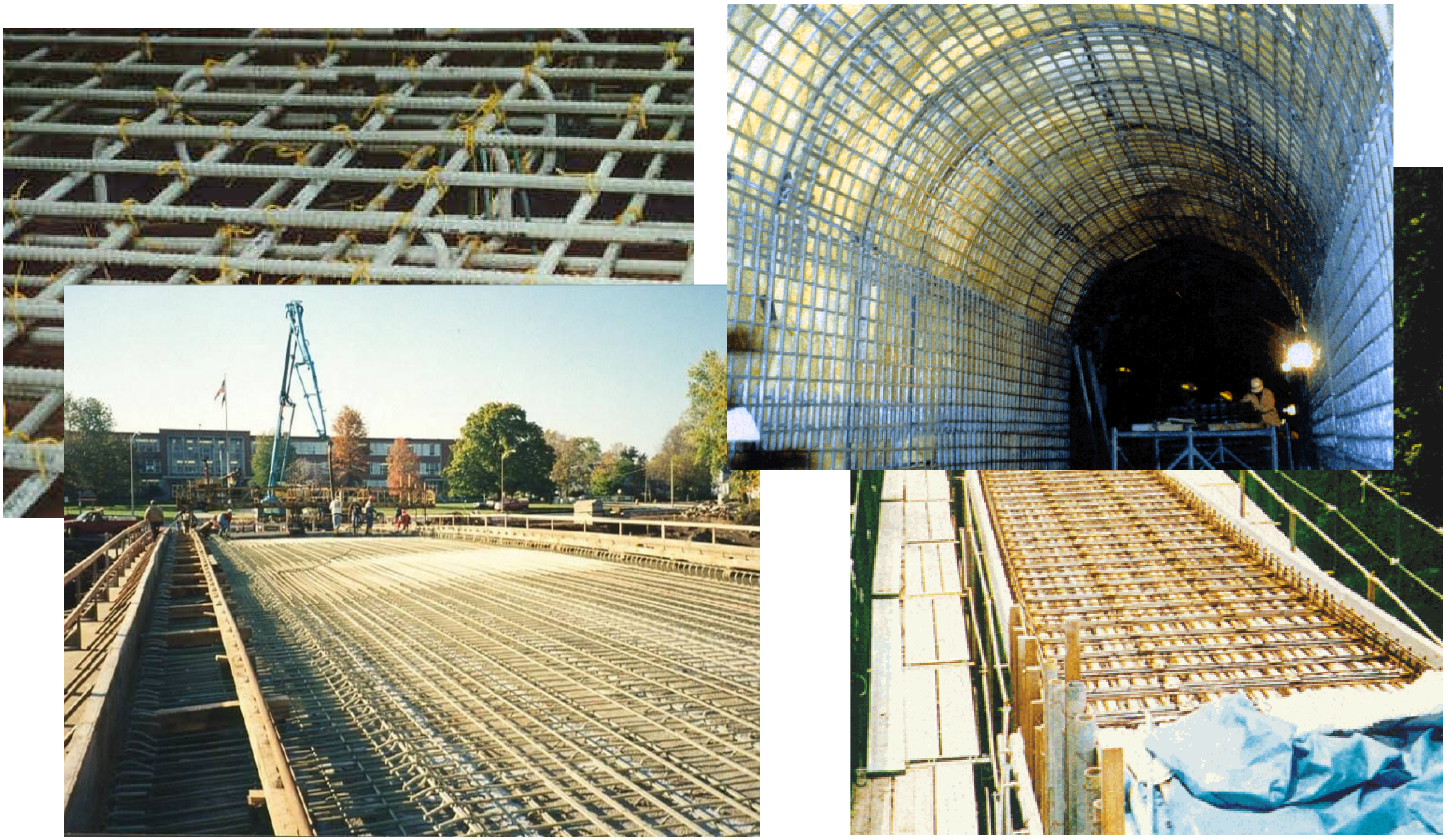






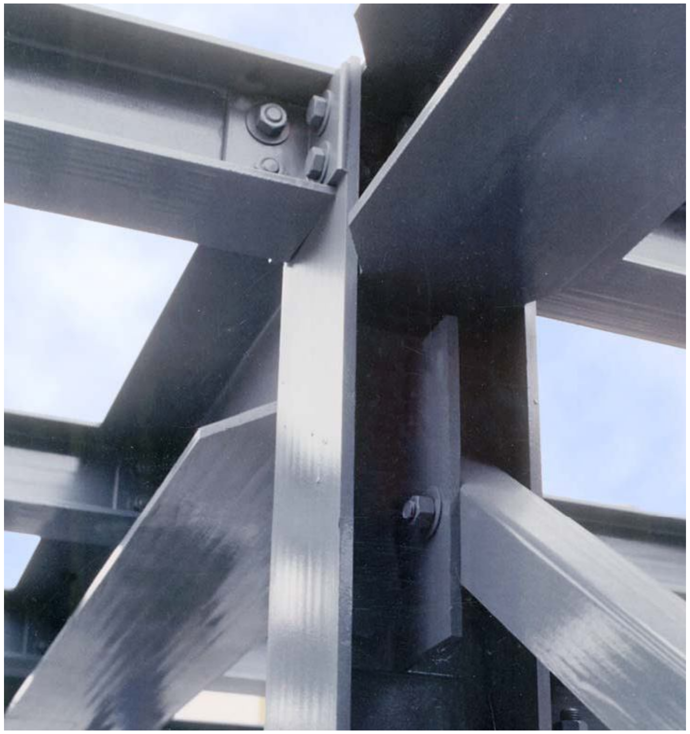
FPR TYPES

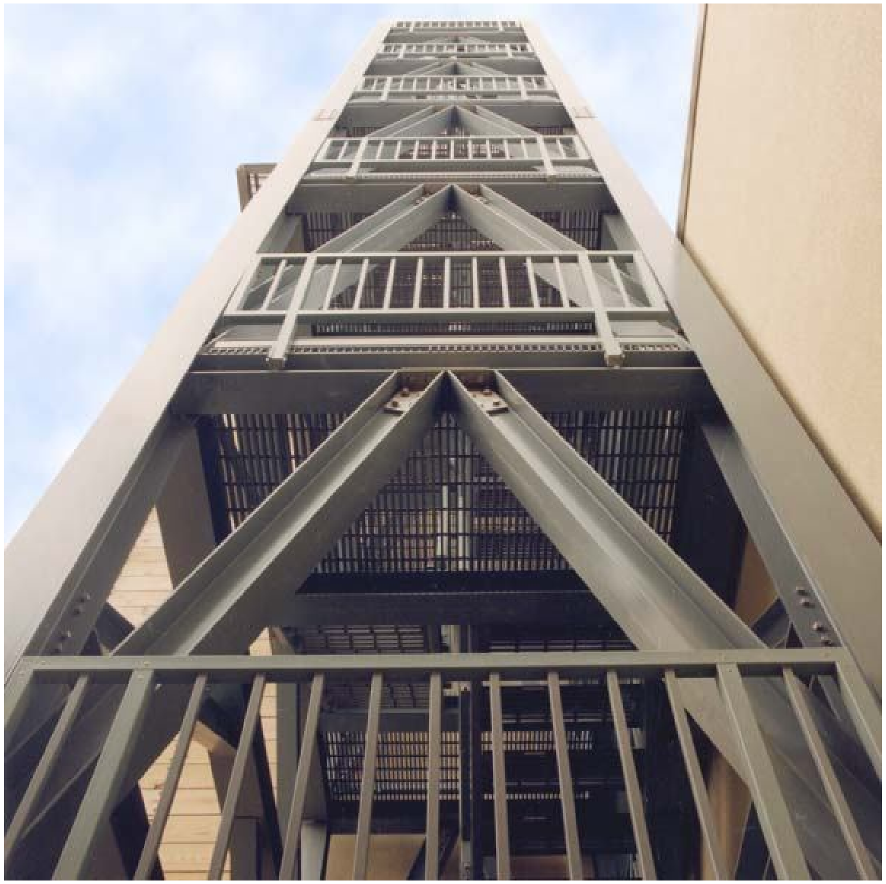
2. REINFORCEMENT → IN CONCRETE



FPR TYPES

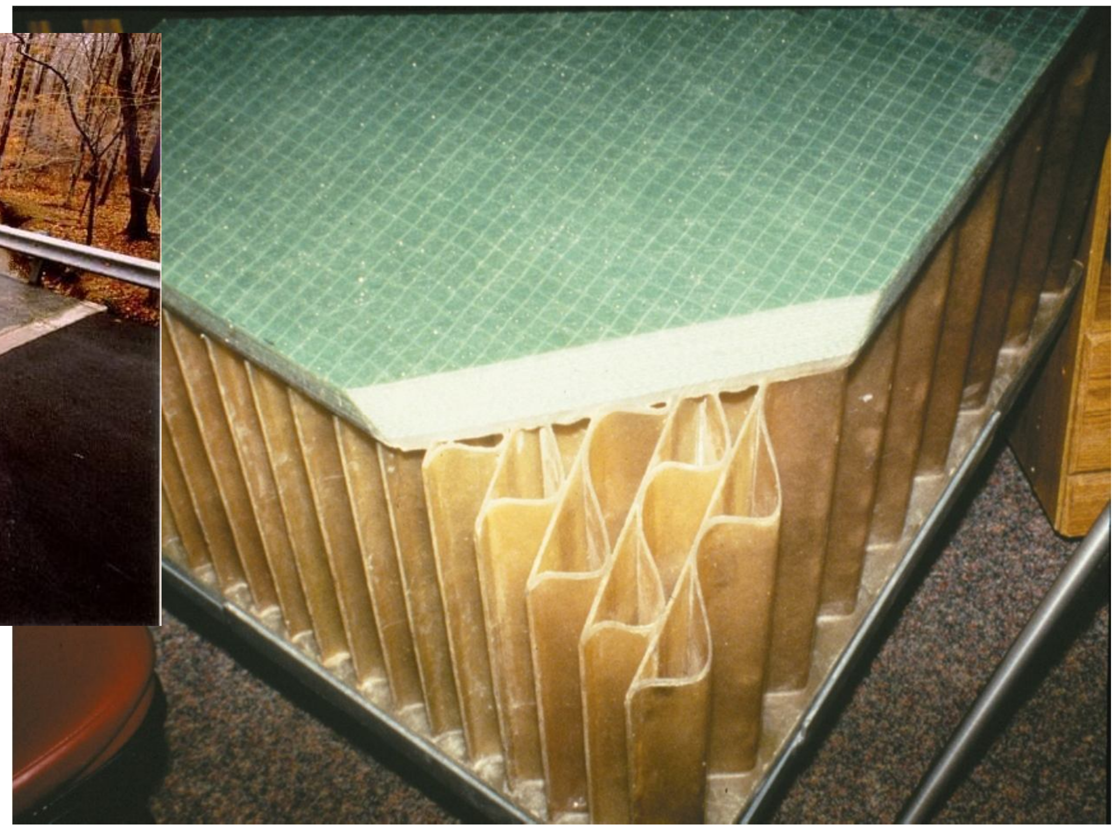
3. PROFILE





TYPES

4. SANDWICH PANELS



STRENGTHENING WITH COMPOSITES

⇒ APPLIED ON THE ELEMENT SURFACE

ADVANTAGES OF THE STRENGTHENING WITH COMPOSITES

- HIGH STRENTHG (~10x steel)
- LESS WEIGHT (~1/4 of steel)
- VERRY THIN
- DURABIITY
- NO MAINTENANCE
- HUGE VARIETY OF SYSTEMS
- FAST EXECUTION
- IMPACT RESISTENCE
- THE MATERIAL PROPRIETIES AS A STRUCTURE

DESADVANTAGIES OF COMPOSITE STRENGTHENING

- LOW FIRE RESISTENCE
- MECHANICAL DETERIORATION
- ULTRAVIOLET DEGRADATION
- SMALLER LIMIT STRAIN AT RUPTURE THAN STEEL
- ELASTIC BEHAVIOUR WITHOUT YIELDING
- RELATIVELY COSTLY

STRANGTHENING WITH COMPOSITES

- REINFORCEMENT CORROSION
- STRUCTURALE CHANGES, NEW SERVICE DEMANDS, MODIFIED CODES
- INCREASE OF THE SAFETY LEVELS
- REDUCING THE EXECUTION DURATION
- DESIGN OR EXECUTION FAULTS

STRENGTHENING SYSTEMS

- WET LAYOUT (FABRIC)

- PREPREG SYSTEMS

- PREFABRICATED SYSTEMS

• *FLEXURAL STRENGTHENING OF RC BEAMS USING CFRP LAMELLAS - ANCHORING SOLUTIONS*

PhDs. Eng. Dan DIACONU
Prof. Dr. Eng. Stoian VALERIU
PhDs. Eng. Sorin Codruţ FLORUŢ

= OBJECTIVE =

=> To clarify some aspects regarded to the influence of some special anchorage and there influences to the overall behaviour of the RC beams subjected to flexure.

= TESTED ELEMENTS =

The beams were cast using Class C32/40 concrete.

⇒ RB – reference beam reinforced with 3 Ø16 bars (ft,med=781 N/mm^2)

⇒ RB2 – second reference beam reinforced with 2 Ø16 bars (ft,med=781 N/mm^2)

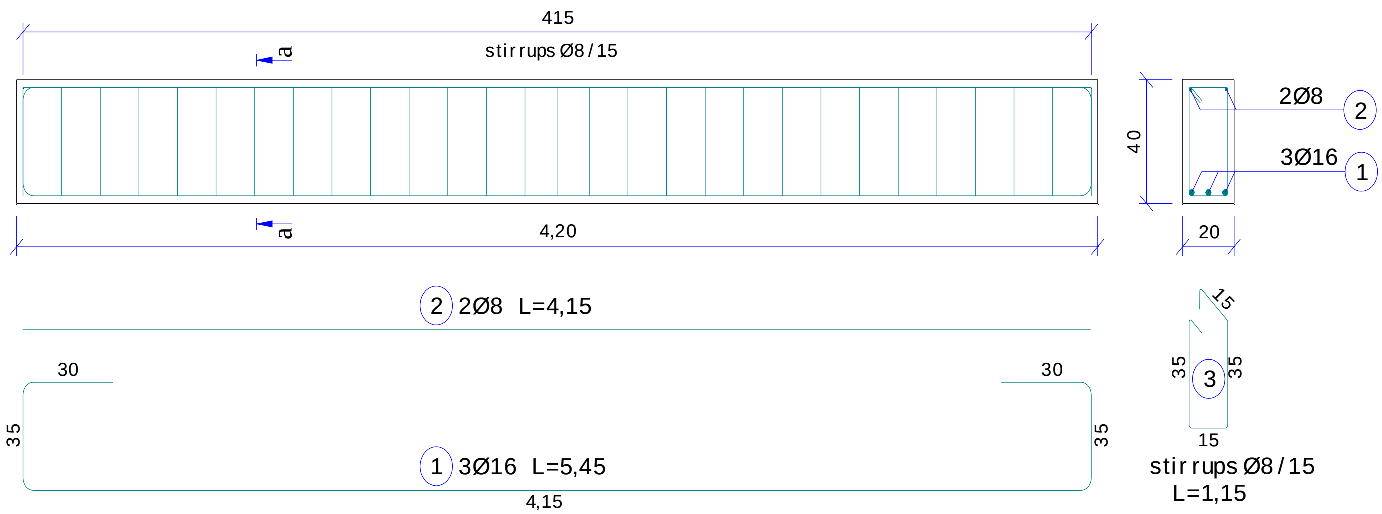
All of the other beams had the same characteristics as beam RB2, the strengthening systems being design so that these beams reach the capacity of RB reference beam.

⇒ R-1S – RC beam strengthened on bottom face with one C-FRP strip 1,2x50 mm in section, anchored at both ends with steel plates (150x20...160 mm) connected to the concrete element by bolts (6 for each steel plate,10 mm diameter)

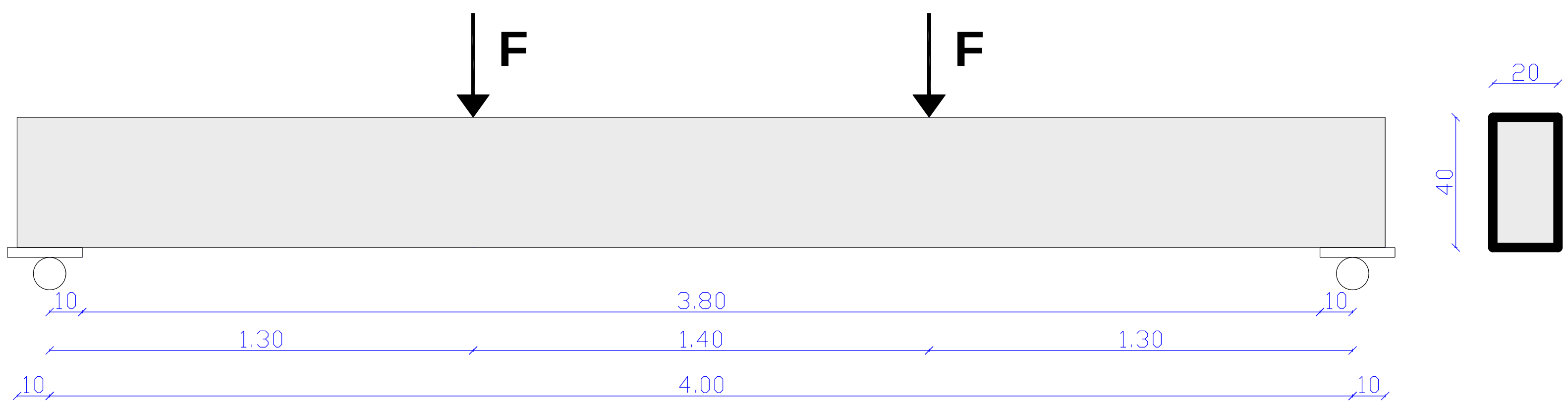
⇒ R-CA-1S – RC beam strengthened on bottom face with one C-FRP strip 1,2x50 mm in section, anchored on its entire length with steel bolts (10 mm diameter) that are chemically anchored in the concrete element

= RB =

RB - reinforcement distribution



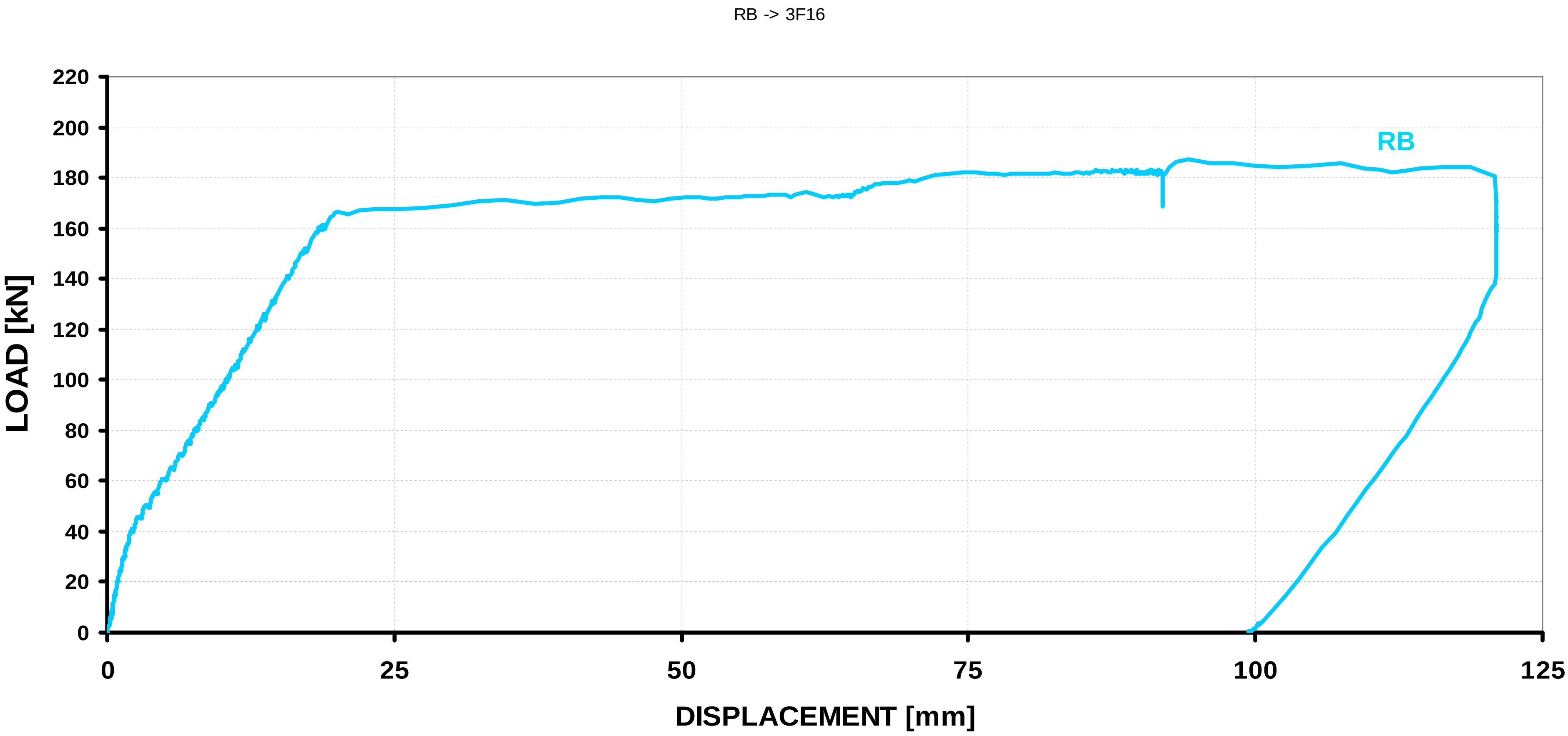
RB - loading scheme



= RB =

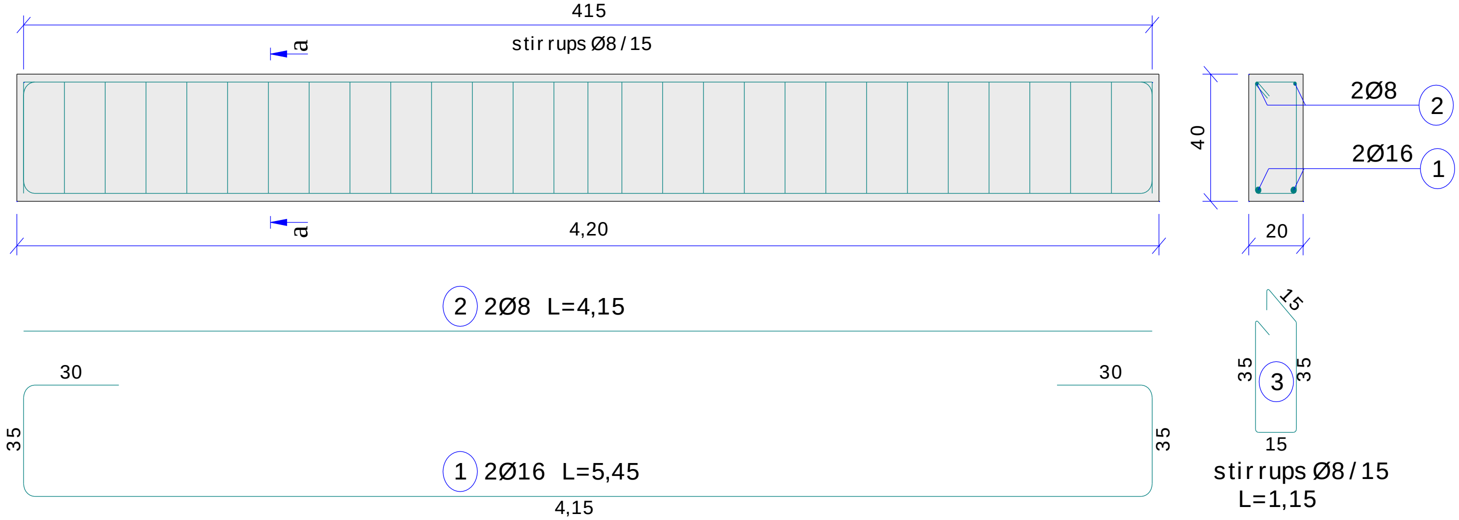
= RB =

The element reaching an ultimate displacement of 121 mm and withstanding a load of 187 kN.

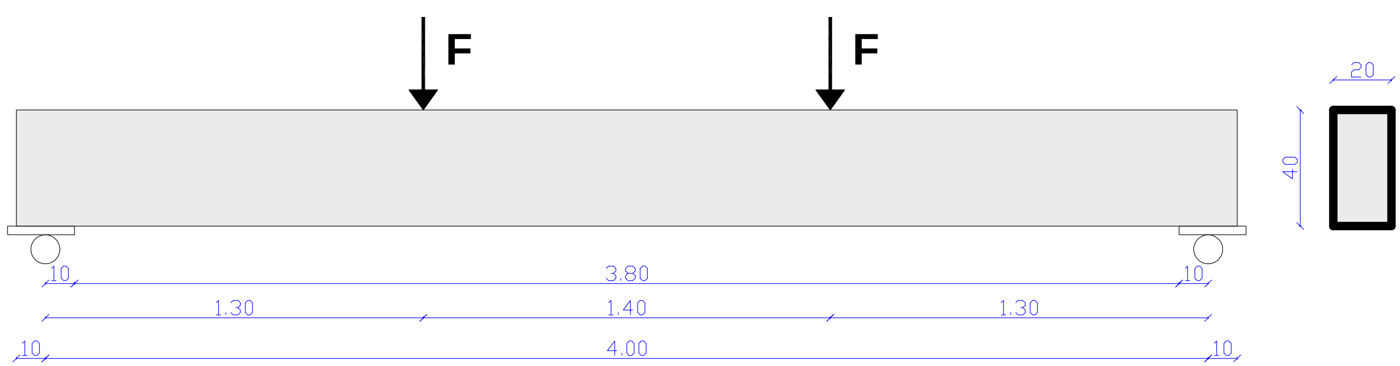


= RB2 =

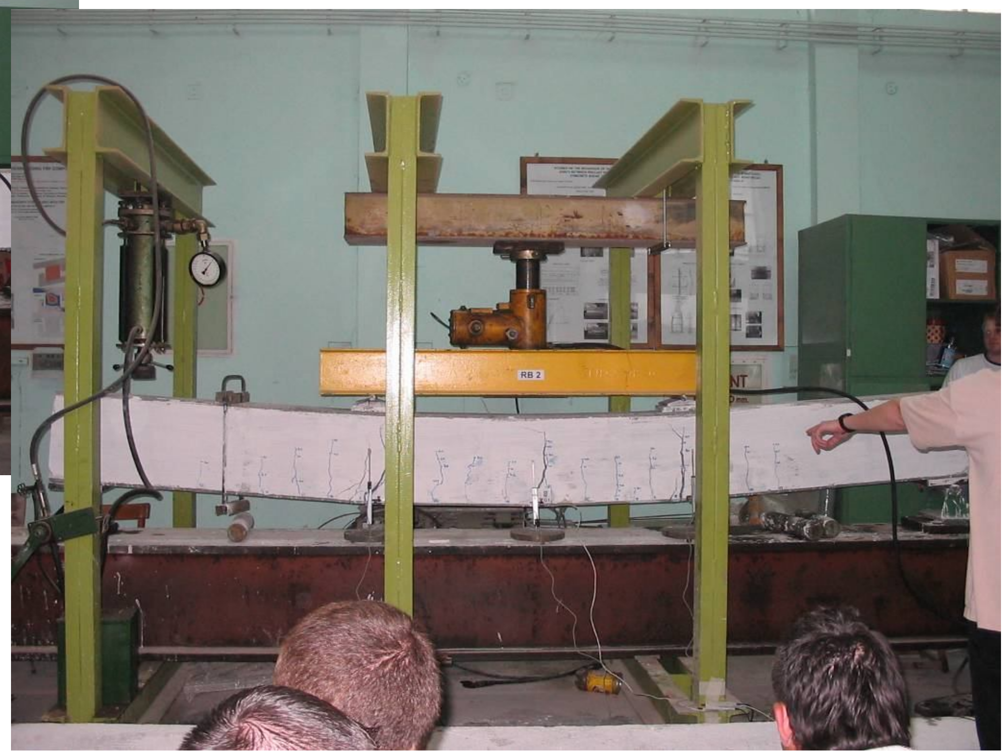
RB2 - reinforcement distribution



RB - loading scheme

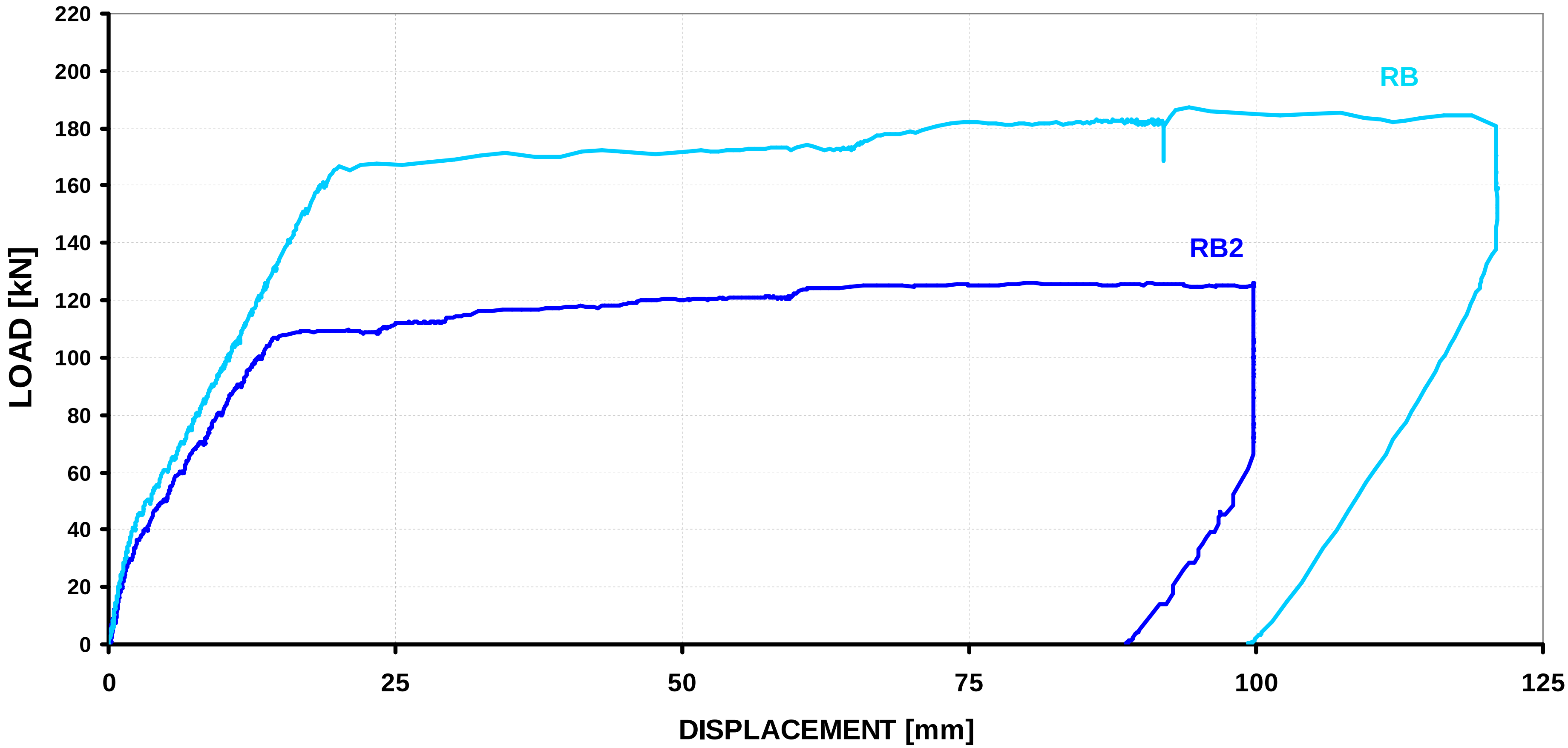


= RB2 =



= RB2 =

The element reaching an ultimate displacement of almost 100 mm and withstanding a load of 126 kN.



= R-1S =

= R-1S =

In order to choose the most adequate epoxy resin used for anchoring the bolts into concrete, a series of pull out tests have been carried out using two different types of resins.

		Resistant torque [Nm]	Failure mode
SIKA	1	over 54	tear out of bolt
	2	over 54	break of bolt
HILTI	1	over 54	break of bolt
	2	over 54	break of bolt

= R-1S =

The steps that need to be carried out for this solution are:

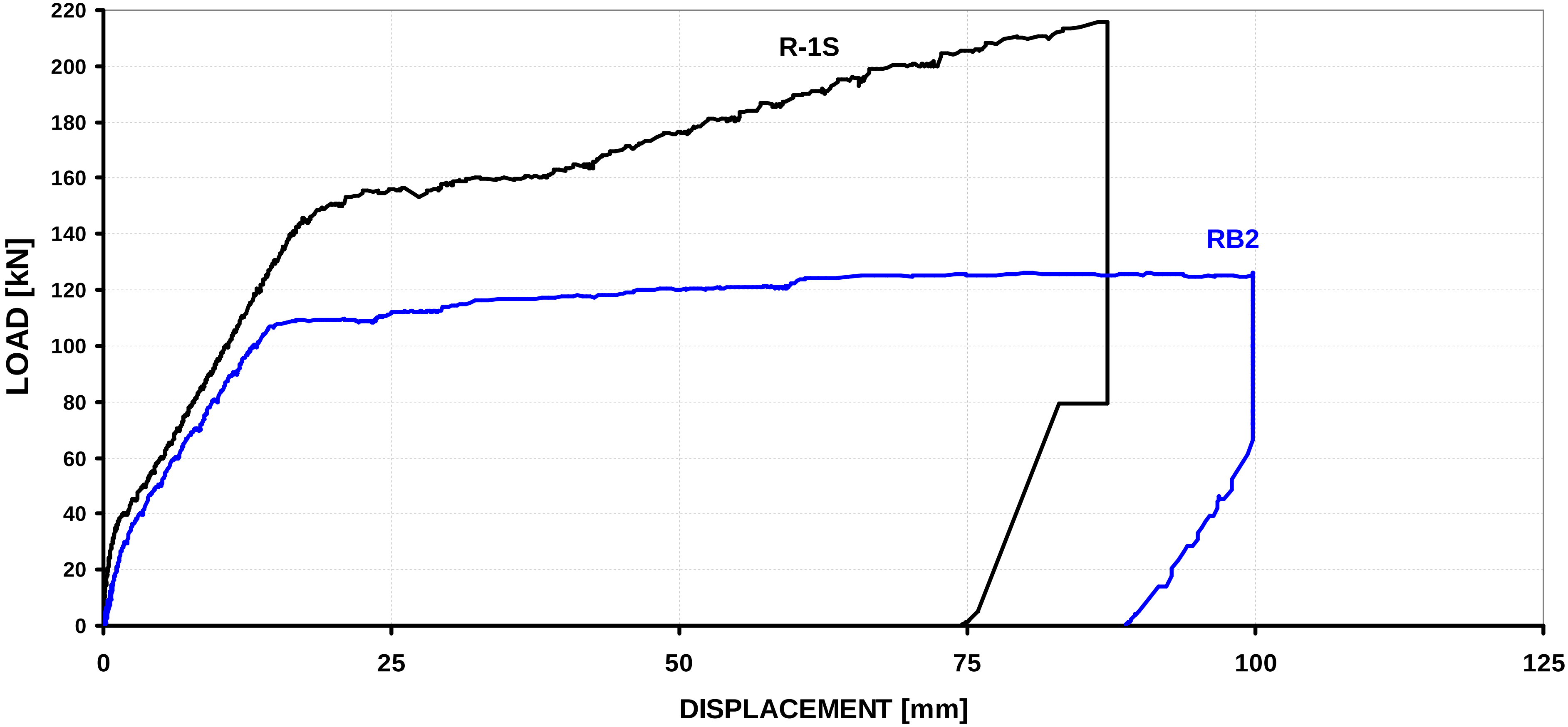
- grinding of concrete surface
- drilling of 12 mm diameter bores for attaching the steel plate to the concrete
- cleaning the dust and all other residues by vacuuming and using compressed air
- applying of epoxy resin on the concrete surface
- applying of epoxy resin on the lamella
- lay up of the lamella and applying pressure with the roller
- filling of the bores with anchoring resin
- inserting of the bolts into the resin filled bores
- applying of epoxy resin onto the steel plates
- lay-up of the steel plates in a proper position so that they fit with the bolts
- tightening the screw by applying a torque of 40 Nm.

= R-1S =

The failure mode of the element began with the composite debonding between the anchorages, followed by concrete crushing in the compression zone, and finally by the sliding out of the FRP from the anchorage simultaneous with the failure of the strip.

= R-1S =

The element reaching an ultimate displacement of 87 mm and withstanding a load of 215 kN.



= R-CA-1S =

= R-CA-1S =

In order to verify the resistance of the lamella to local pressure on the hole, a series of tension tests have been carried out :

- solely the lamella – for reference
- with steel plate of 2 mm in thickness glued with resin on one face
- with steel plate of 2 mm in thickness glued with resin on one both faces
- with GFRP fabrics wrapped around the lamella in a double layer
- with GFRP fabrics glued only on one face in four layers (0-90-45-135)

= R-CA-1S =

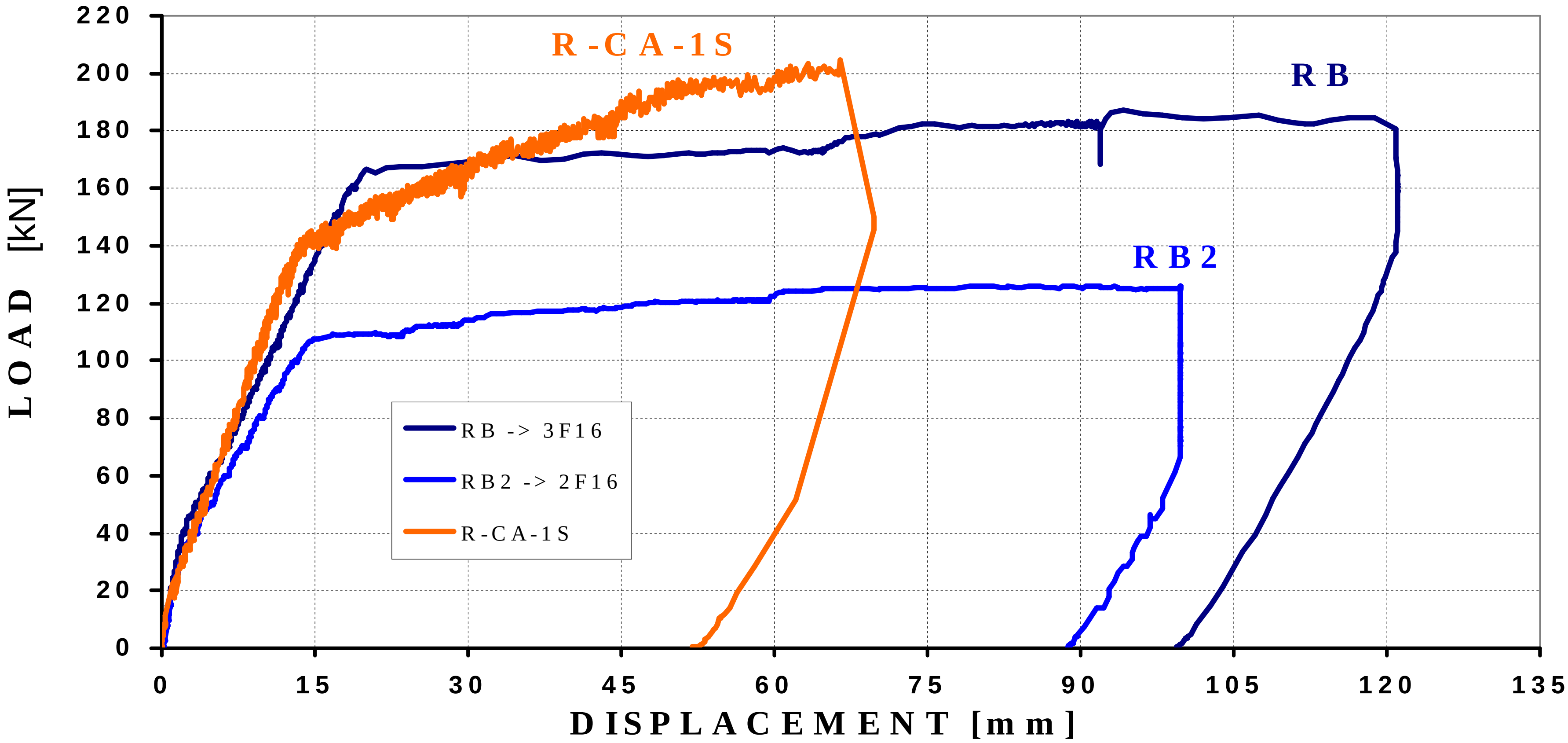
The steps that need to be carried out for this solution are:

- drilling of 12 mm diameter bores for attaching the steel plate to the concrete
- cleaning the dust and all other residues by vacuuming and using compressed air
- creating 10 mm holes in the lamella
- positioning the lamella on the RC beam
- filling of the bores with anchoring resin
- fitting the bolts to match the holes in the lamella
- placing the washers
- tightening the screws

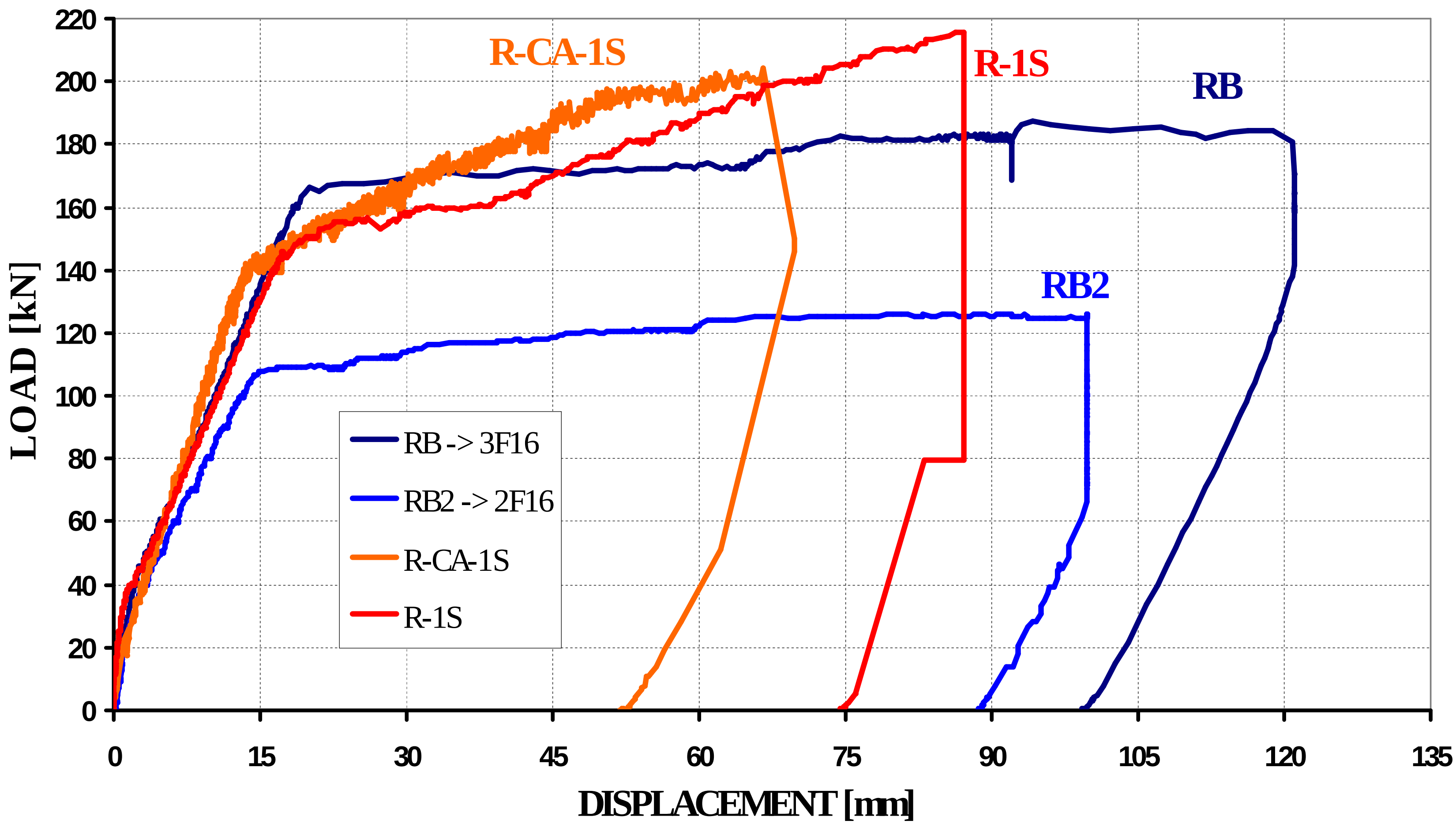
The element failed by concrete crushing in the compressed zone simultaneously with the rupture of CFRP lamella at the mid span.

= R-CA-1S =

The element reaching an ultimate displacement of almost 70 mm and withstanding a load of 204 kN.



= COMPARATIVE LOAD – DISPLACEMENT DIAGRAM =



= CONCLUSIONS =

The two beams strengthened by the above mentioned methods have resisted to loads higher than the designed one - RB (187 kN).

The ultimate load capacity for element R-1S was higher with 15% than that of RB beam, but ultimate reached displacement was 28% lower than in the case of RB element.

The ultimate load capacity for element R-CA-1S was higher with 9% than that of RB beam, but ultimate reached displacement was 30% lower than in the case of RB element.

In terms of structural efficiency the first solution is slightly superior, but in terms of easiness of application it is inferior to the second solution.

The second solution (R-CA-1S) limits the use of epoxy resin only to anchoring the bolts into the concrete beam, significantly reducing the exposure to toxic environment effects that it creates while conducting the strengthening procedure.

The execution and curing time it is lower for the second solution in comparison with first one (one day for R-CA-1S instead of one week for R-1S).

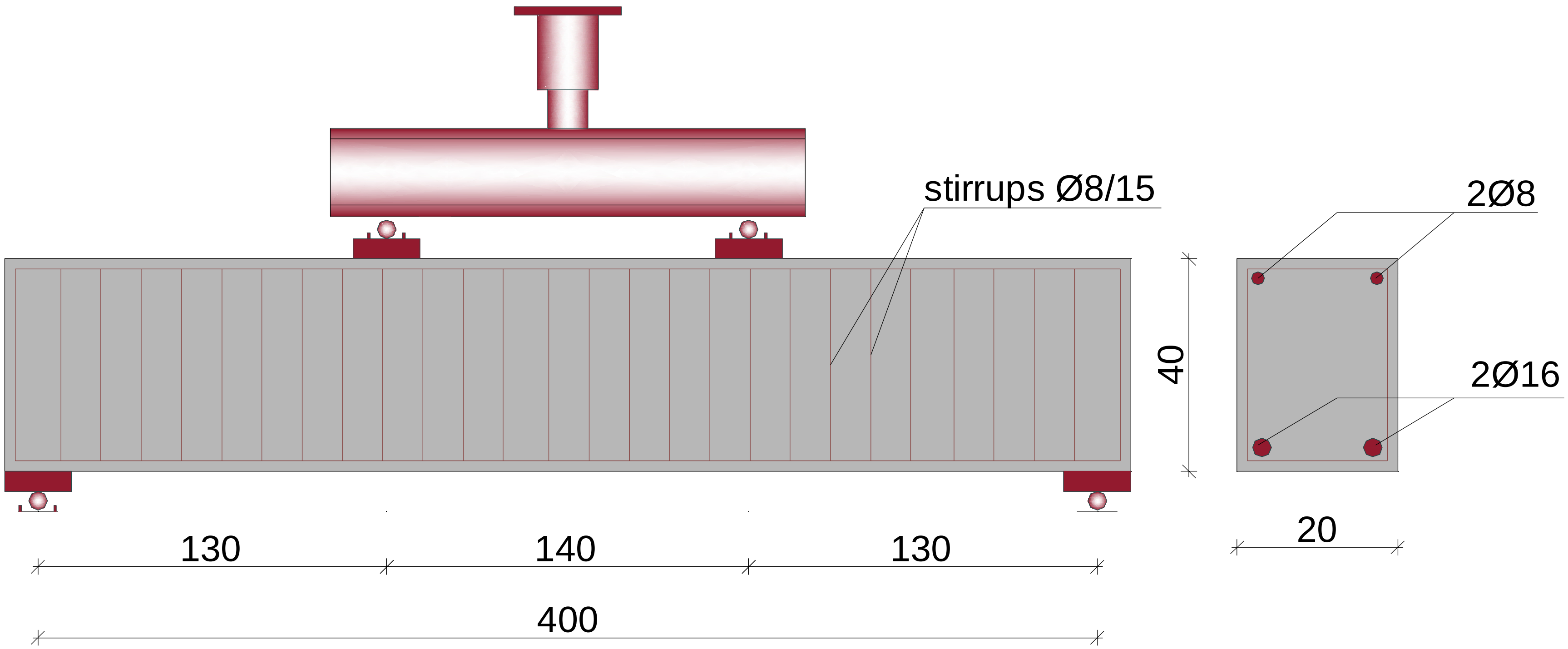
As it can be seen in the comparative load-displacement diagram, the strengthened elements presented a slightly higher initial stiffness than the reference beams, but were characterized by a poorer ductility.

II. RC BEAMS STRENGTHENED WITH FRPS → DIMENSIONING

CROSS SECTION:	20 x 40 CM
SPAN:	400 CM
REFERENCE BEAM 1 (RB):	3ϕ16
REFERENCE BEAM 2 (RB2):	2ϕ16
RETROFITTED SPECIMENS:	2ϕ16

⇒ FOR STRENGTHENING DIFFERENT FRP COMPOSITES SYSTEMS WERE USED, WHICH WERE DESIGNED TO REACH AN APPROPRIATE LOAD BEARING CAPACITY AS THE FIRST REFERENCE BEAM (RB).

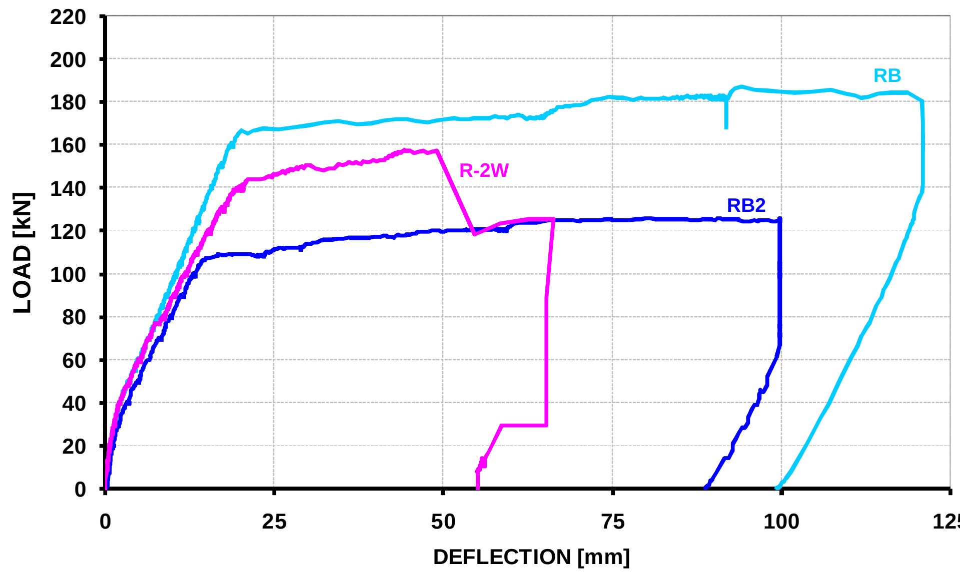
II. RC BEAMS STRENGTHENED WITH FRPS → TEST SET-UP



II. RC BEAMS STRENGTHENED WITH FRPS → TEST RESULTS

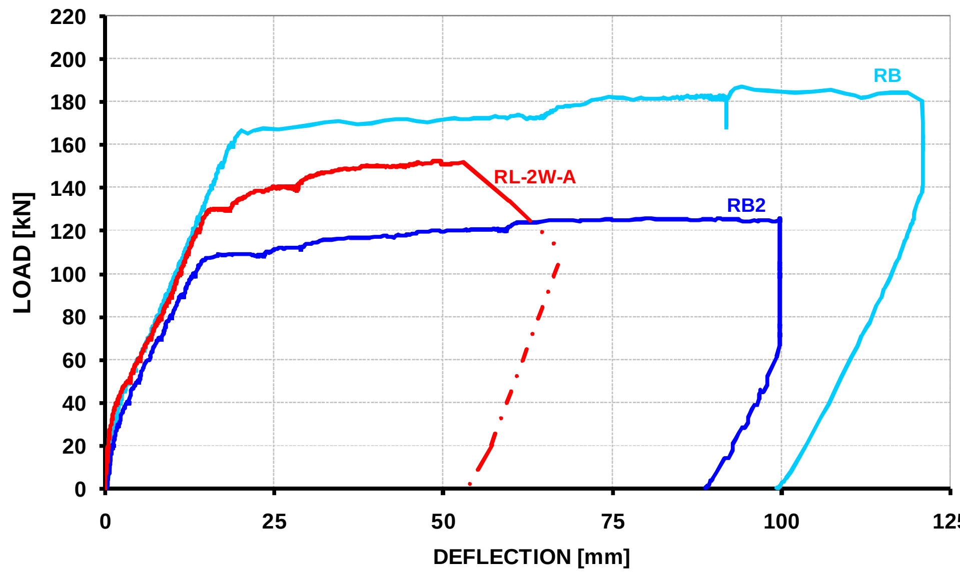
R-2W → strengthened with 2 layer of 18cm wide unidirectional carbon FPR fabric in the bottom part, without any anchor system.

→ The failure of the element was through debonding of the FRP.



II. RC BEAMS STRENGTHENED WITH FRPS → TEST RESULTS

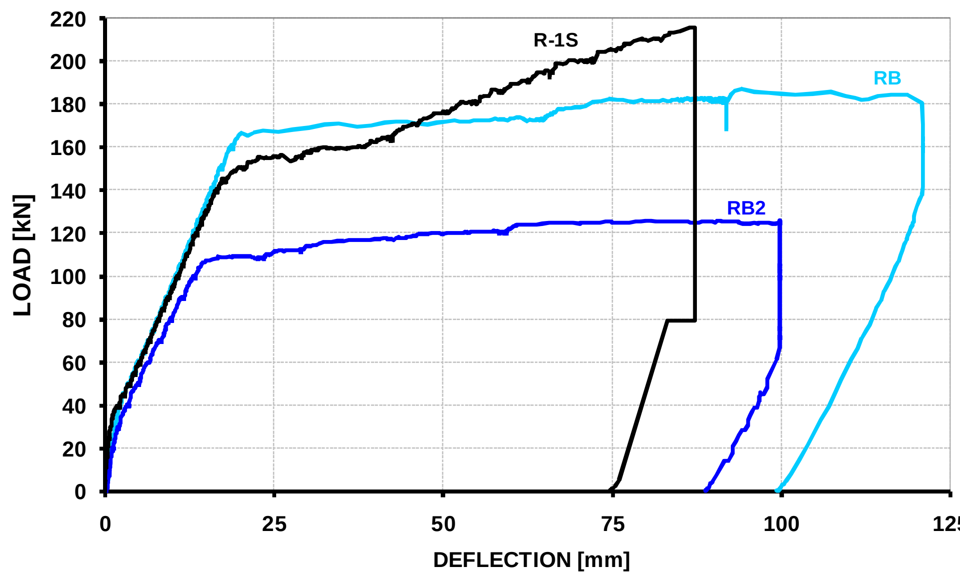
RL-2W-A → strengthened with 2 layer of 9cm wide unidirectional carbon FPR fabric applied in the lateral part of the both side, anchored with three spikes/end/side.
→ The failure of the element was debonding inside the FRP.



II. RC BEAMS STRENGTHENED WITH FRPS → TEST RESULTS

R-1S → strengthened with 1 layer of 5cm wide carbon FPR strip applied in the bottom part and anchored at the ends with steel plates.

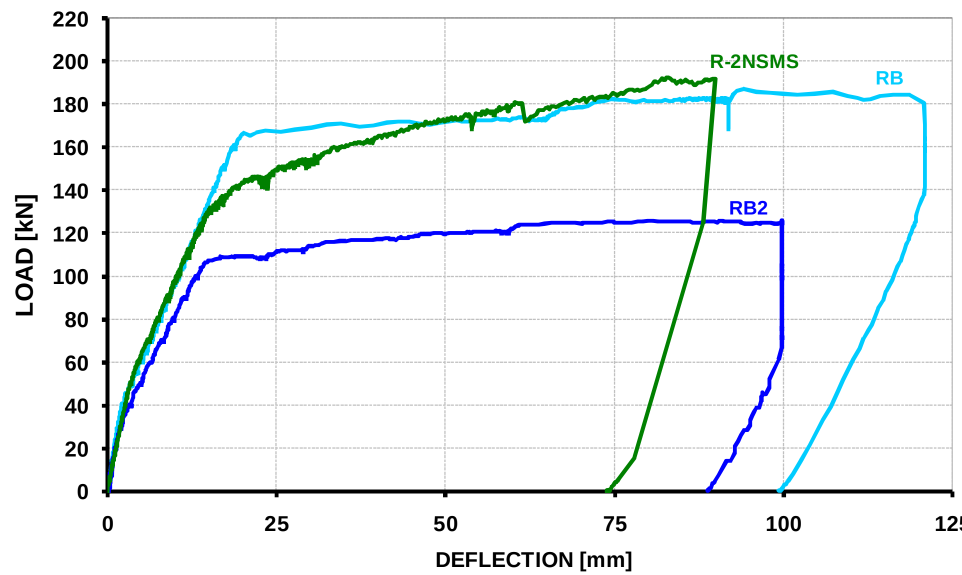
→ the failure began with the composite debonding between the anchorages, followed by concrete crushing in the compression zone, and finally by the sliding out of the FRP from the anchorage simultaneous with the failure of the strip.



II. RC BEAMS STRENGTHENED WITH FRPS → TEST RESULTS

R-2NSMS → strengthened with two 20mm wide carbon FPR strip, in the bottom part of the beam, applied in cut grooves (NSM technique).

→ The failure began with the composite debonding in the grooves, followed by concrete crushing in the compression zone, and finally by the tensile failure of the FRP strip in the middle region of the beam.



II. RC BEAMS STRENGTHENED WITH FRPS → CONCLUSIONS

→ BASED ON THE PERFORMED EXPERIMENT, RESPECTIVELY ON THE BEHAVIOUR OF THE TESTED SPECIMENS, THE FAVOURABLE EFFECTS OF MECHANICAL AS WELL AS CHEMICAL ANCHORAGE WERE EXPERIMENTALLY DEMONSTRATED, BOTH FOR BOTTOM AND LATERALLY APPLIED COMPOSITES.

→ THE MOST EFFECTIVE STRENGTHENING SOLUTION FOR THE INCREASING OF THE FLEXURAL CAPACITY OF REINFORCED CONCRETE BEAM PROVED TO BE THE NSM TECHNIQUE

Shear strengthening
of RC beams in Timisoara

Shear strengthening
CASE STUDY

- RC FRAME STRUCTURE
- DESIGNED IN *1964-1981 (1978)*
- LEVEL NR.: 3
- FUNCTIONALITY: TYPOGRAPHY

⇒ **PROBLEMS: SHEAR CRACKS IN SEVERAL BEAMS**



Shear strengthening
CASE STUDY

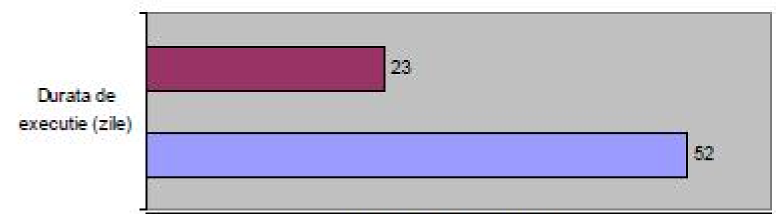
STRENGTHENING PROPOSALS

1. CFRP COMPOSITES → U WRAP

2. CLASSICAL METHOD → additional reinforcement and shotcrete

	Pret materiale (euro)	Manopera (euro)	Pret final (euro)
camasuire	3245	1814	5059
tesatura CFRP	4505	630	5135

tesatura CFRP · camasuire



	Durata de executie (zile)
tesatura CFRP	23
camasuire	52

camasuire · tesatura CFRP

ECONOMICAL STUDY

Shear strengthening
CASE STUDY

MAIN ADVANTAGES:

- No needs for interventions on the recently rehabilitated roof
- Beams cross section maintained, esthetic was not effected
- No significant interruption in the production process
- Execution time more less than for the classic method (with 55%)
- Simple process

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Surface cleaning

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Surface cleaning

Preparing the anchorage zone

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

The clean surface ready to strengthening

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

The clean surface ready to strengthening

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Crack injection

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Surface preparation

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Surface preparation

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Sealing and saturating the surface

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Applying the fibers

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Applying the fibers

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Applying sand
to improve bond between the FRP and the finishing mortar

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Applying top coat

Shear strengthening
CASE STUDY

STRENGTHENING PHASES:

Applying top coat

Shear strengthening
CASE STUDY

THANK YOU FOR YOUR ATTENTION !

Ductility Increasing for Concrete Columns: Experimental Results

- Authors: C. Al. Dăescu, V. Stoian, T. Nagy-György, D. Dan, I. Demeter

-

- **Strengthening of the RC columns**
- **Superposition of the two methods of strengthening (bending and confinement)**
- **Study on the ductility of the strengthened specimens.**

Testing program – 1st series:

Element	Consolidation system	Test type
C1M	[redacted]	[redacted]
C1C	[redacted]	[redacted]
C1M – CW – BC	[redacted]	[redacted]
C1C – CW – BC	[redacted]	[redacted]
C2M – GW – BC	[redacted]	[redacted]
C2C – GW – BC	[redacted]	[redacted]
C3M – BM – AF	[redacted]	[redacted]
C3C – BM – AF	[redacted]	[redacted]
C4M – BM + CW	[redacted]	[redacted]
C4C – BM + CW	[redacted]	[redacted]

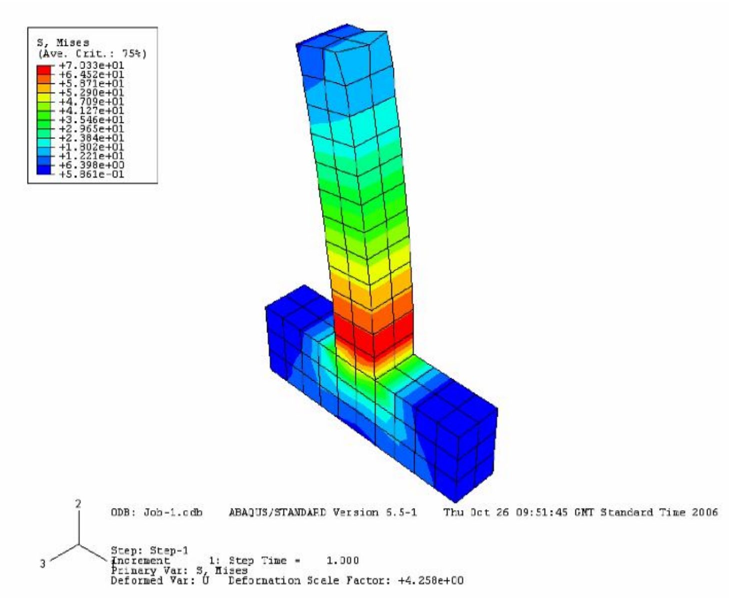
[redacted]

Testing program – 2nd series:

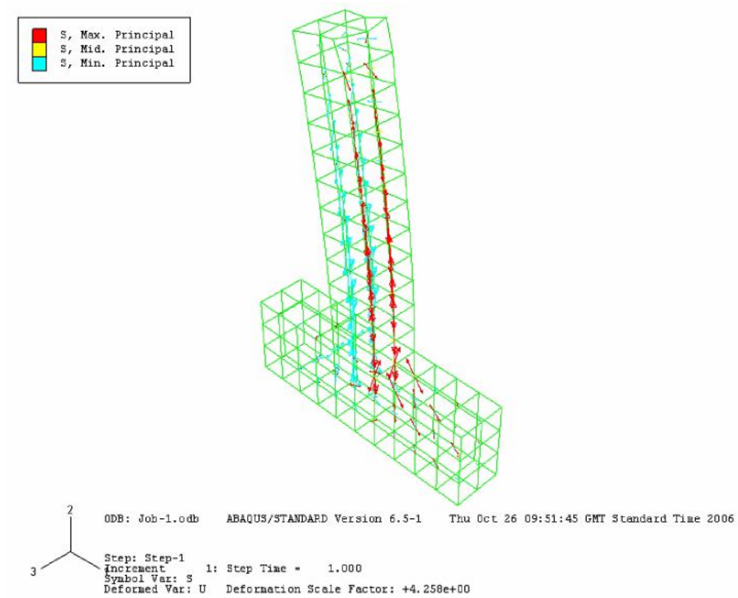
Element	Consolidation system	Test type
[redacted]		

[redacted]

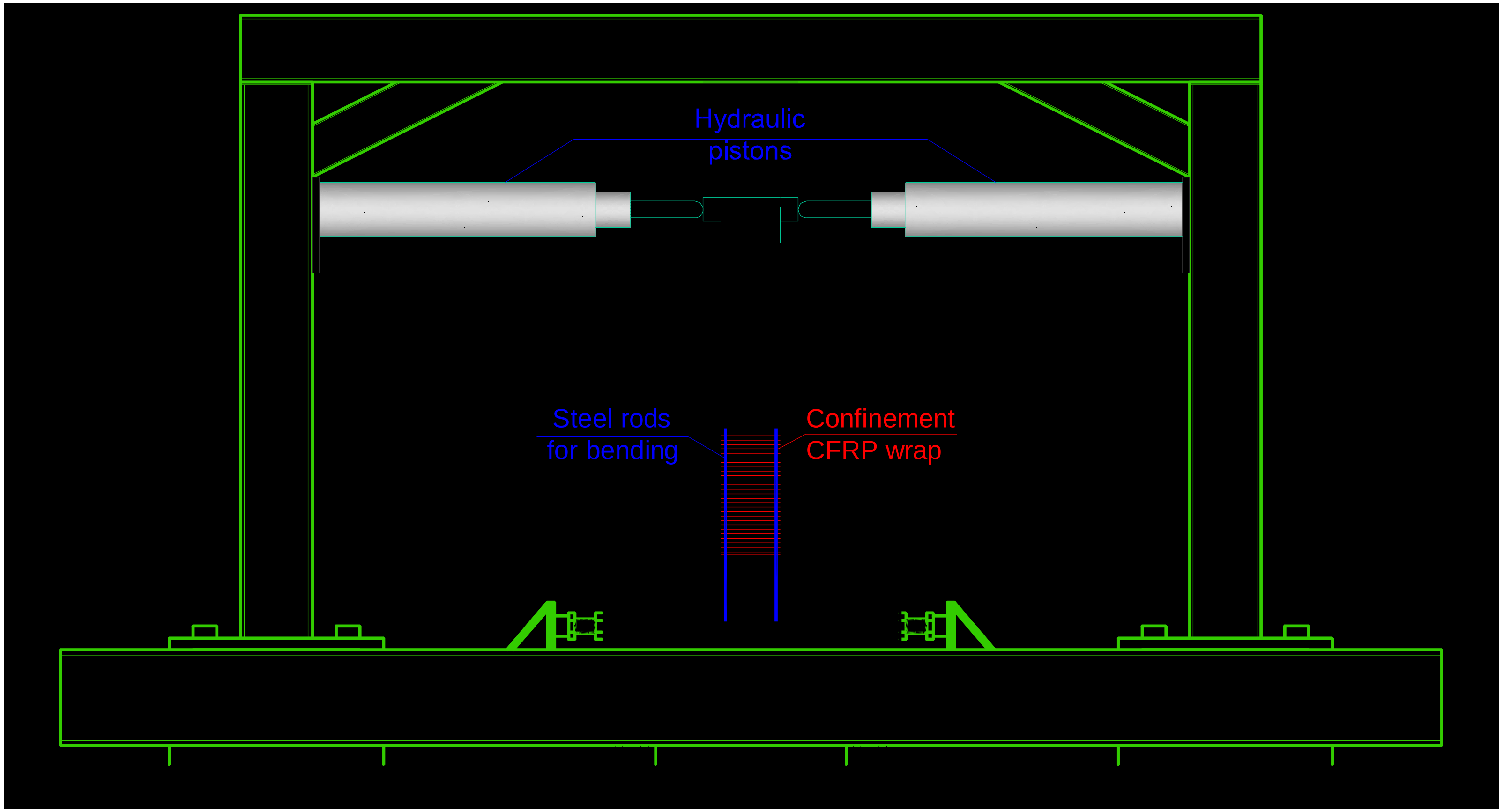
Abaqus initial model in monotonic loading



Magnitude of principal stresses

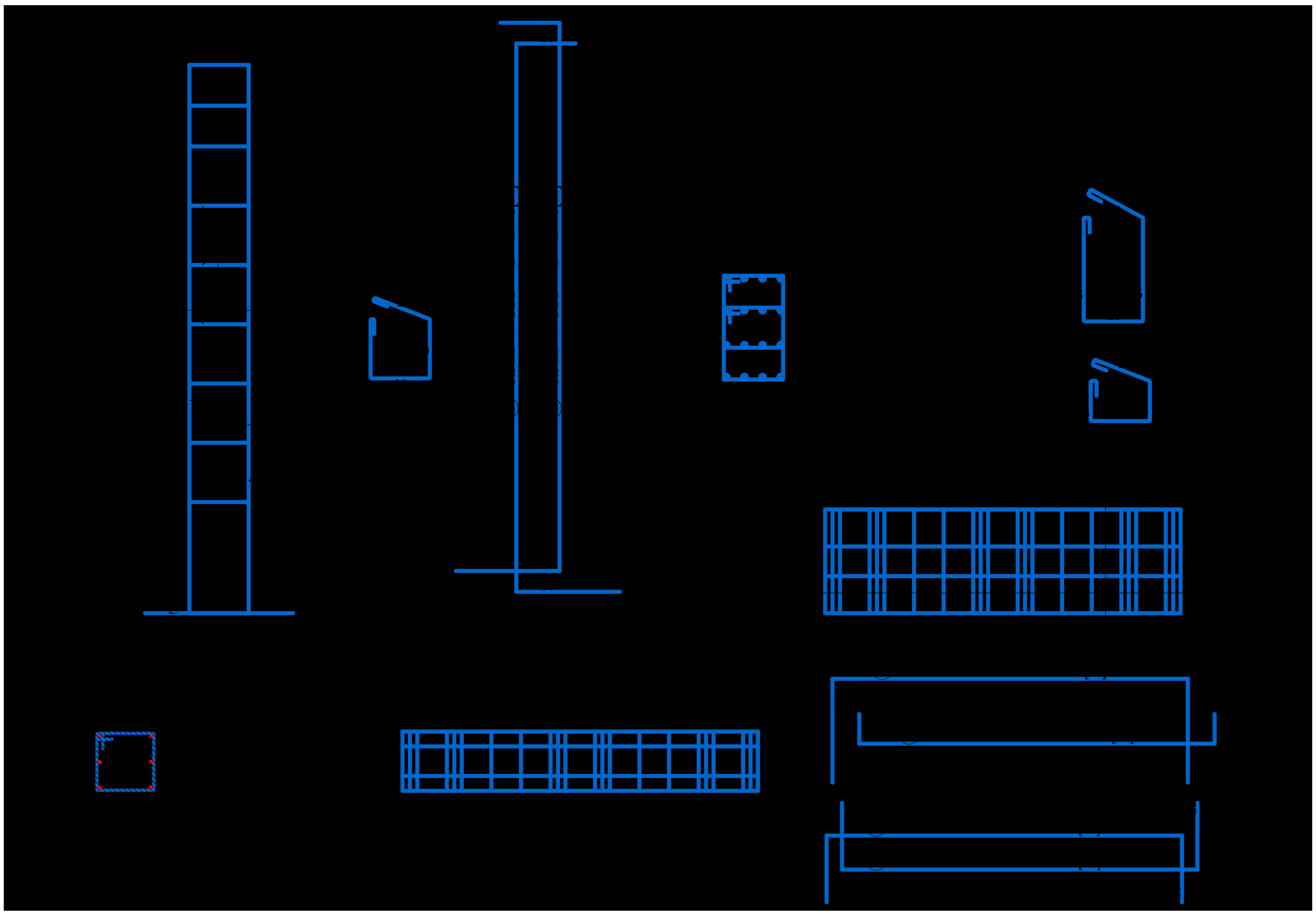


Direction of principal stresses

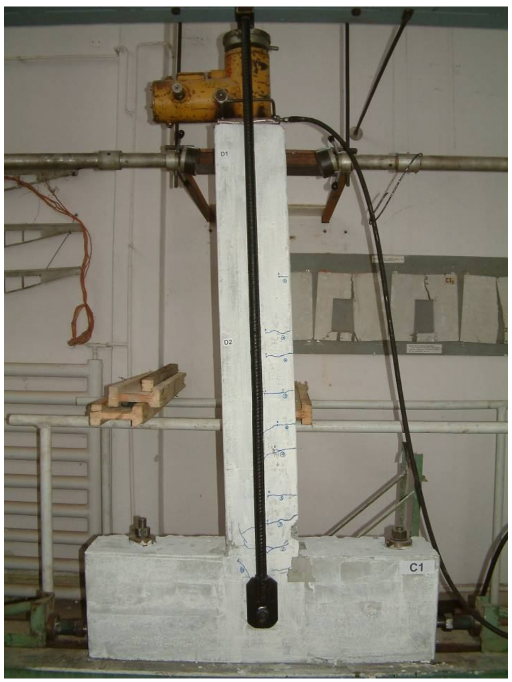


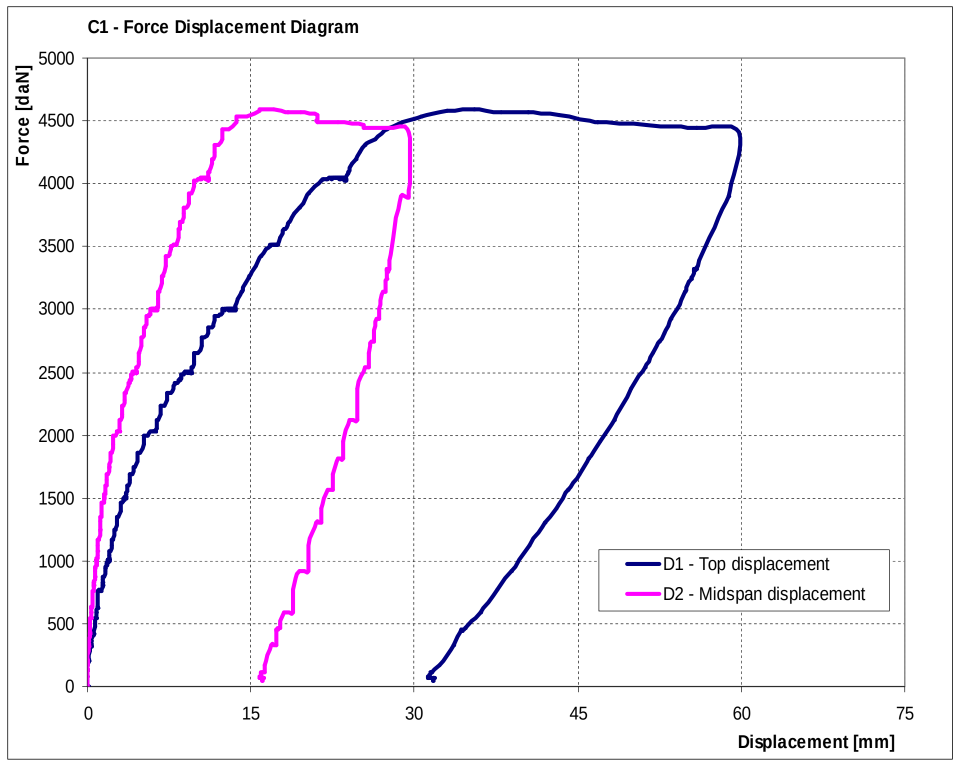
Layout of the testing frame. No axial force applied.

Reinforcing Layout of the Specimens

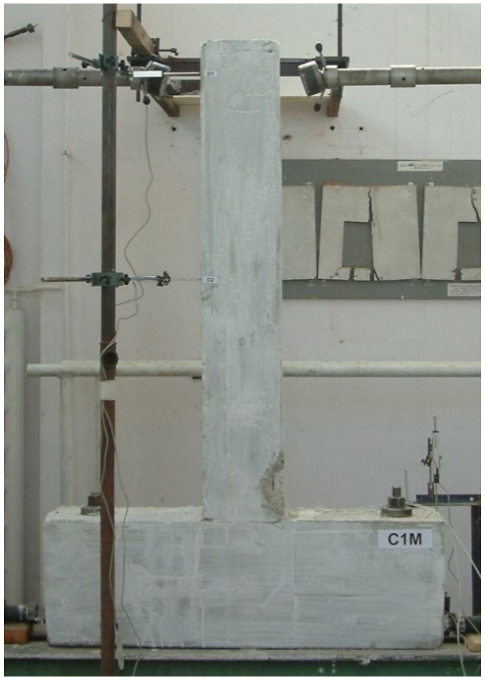


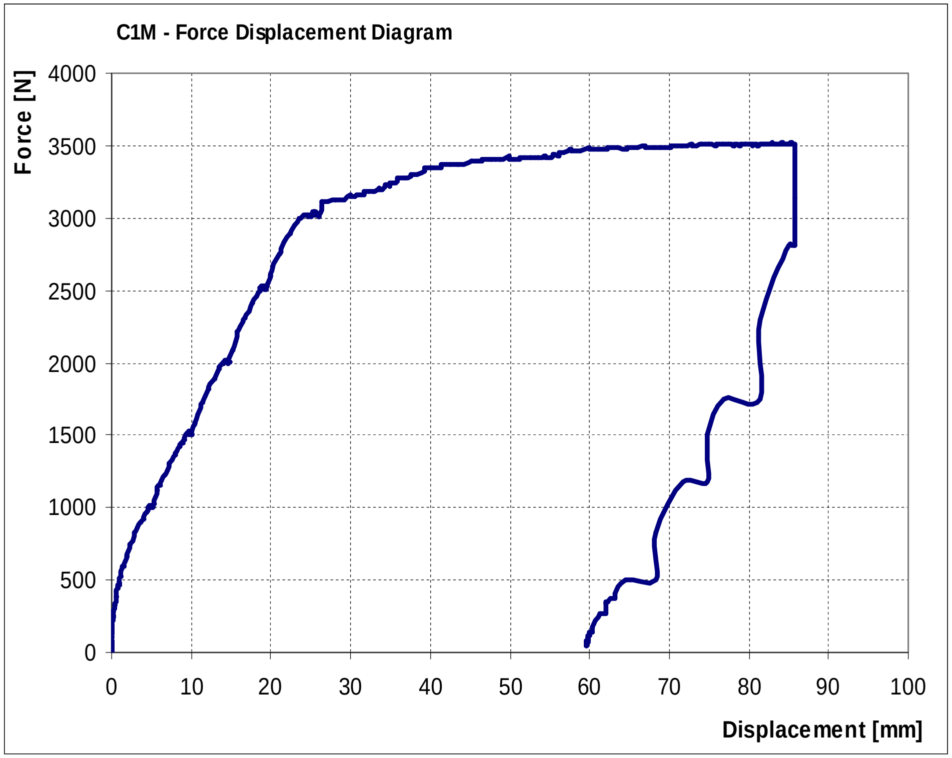
Reference specimen [C1]
Monotonic loading, with axial force



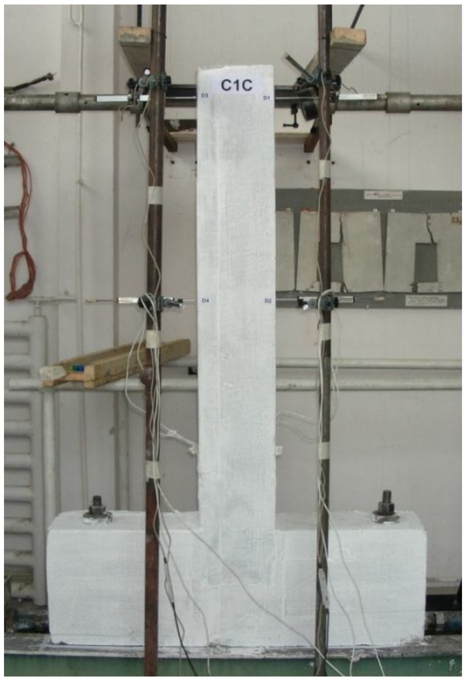


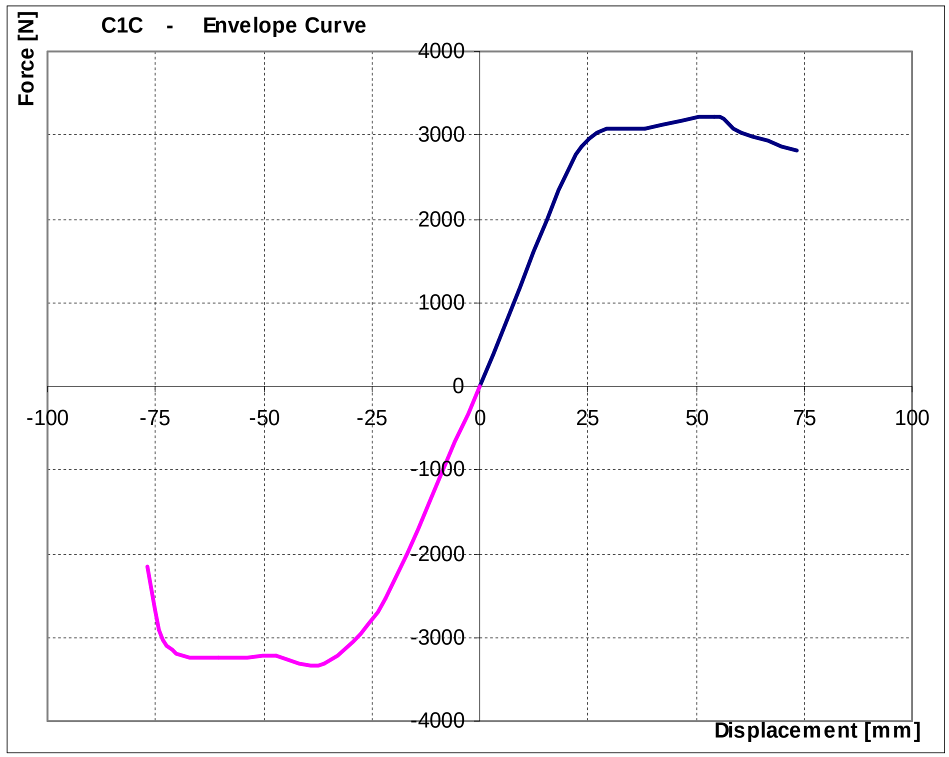
Reference specimen [C1M]
Monotonic testing



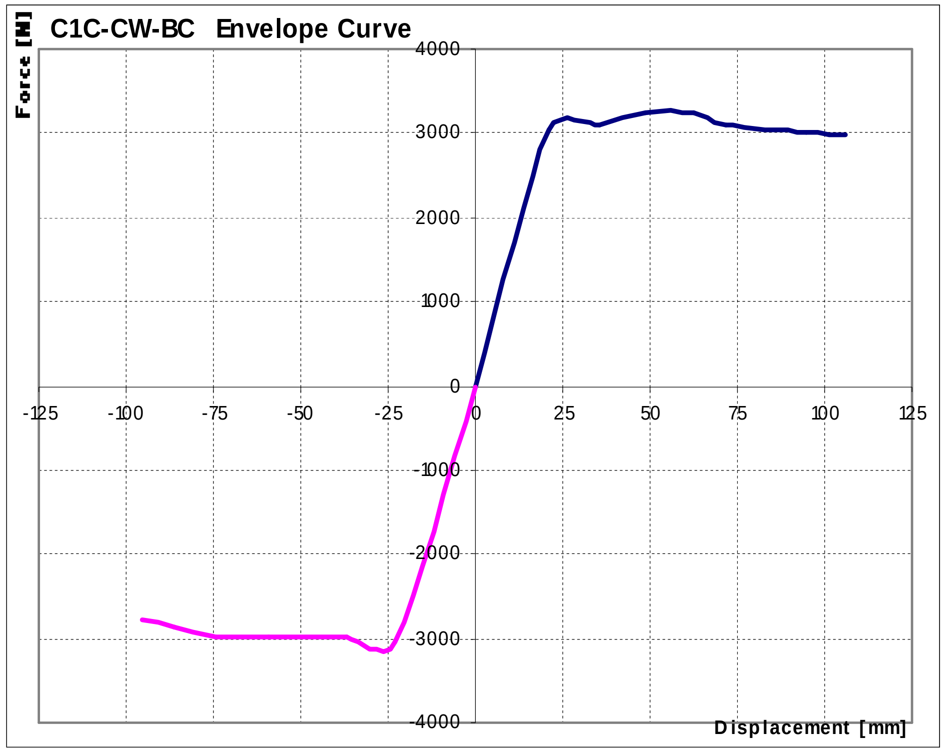


Reference specimen [C1C]
Cyclic testing

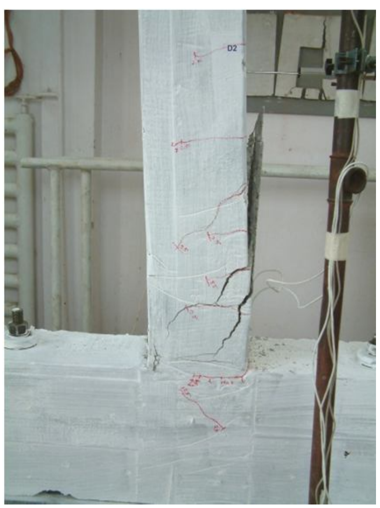


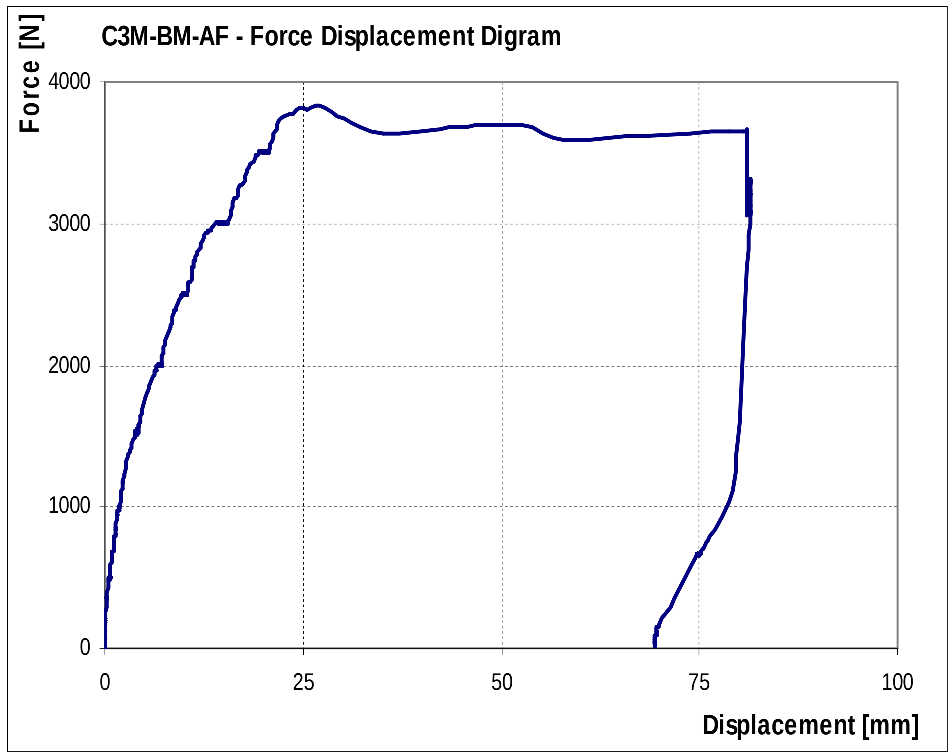


Specimen strengthened by a carbon fiber wrap base confinement [C1C-CW-BC] Cyclic testing

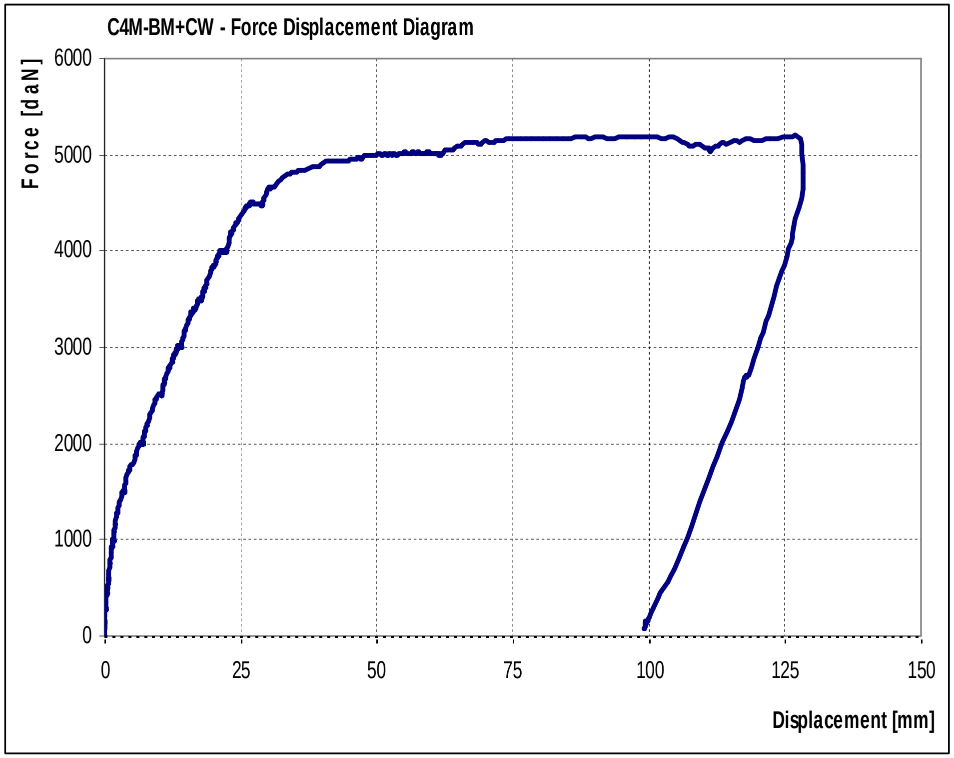


Specimen strengthened using metallic rods anchored into the foundation. [C3M-BM-AF] Monotonic testing

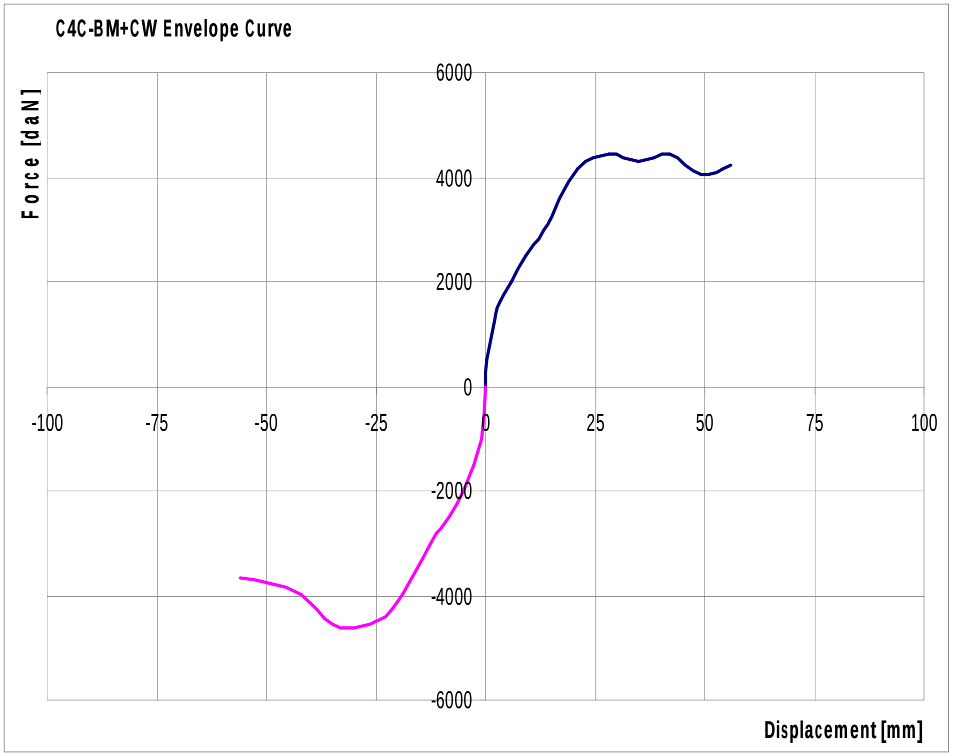




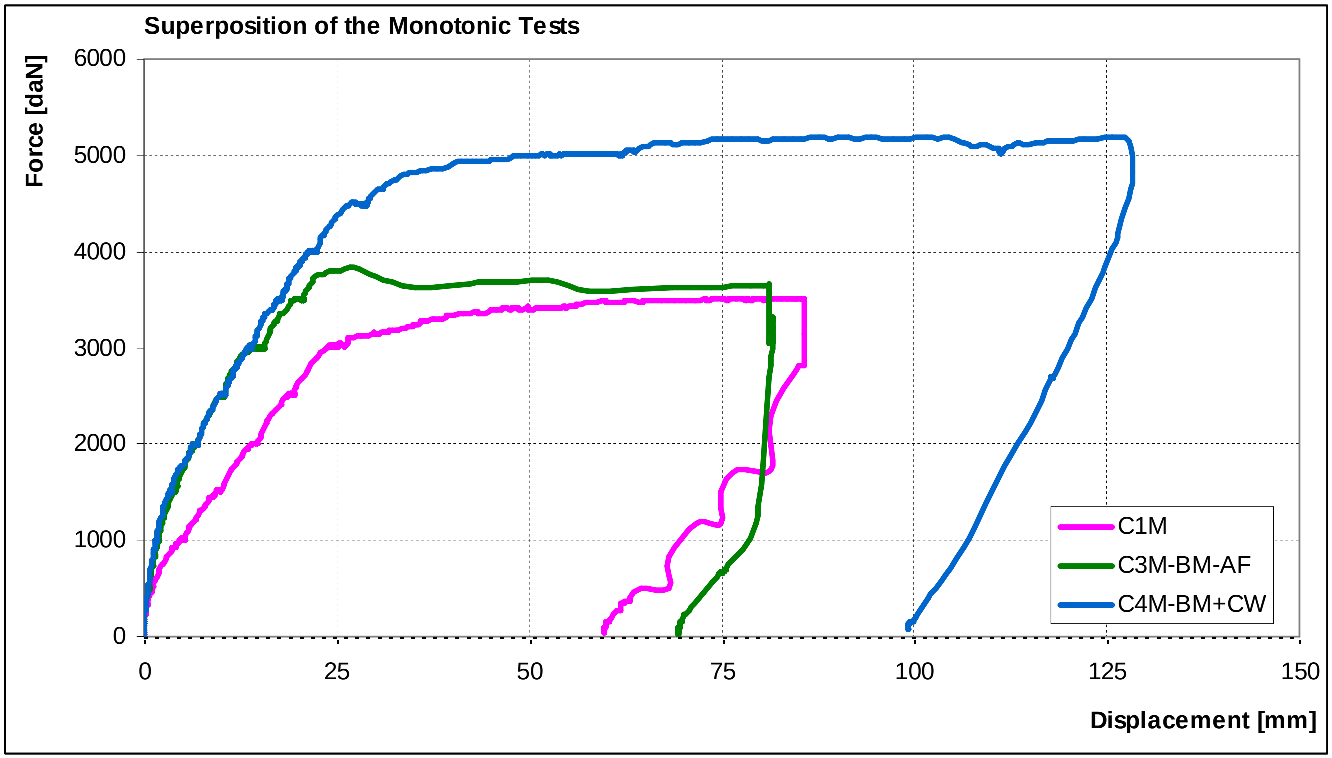
Specimen strengthened using metallic rods anchored into the foundation and a carbon fiber wrap base confinement [C4M-BM+CW] Monotonic testing



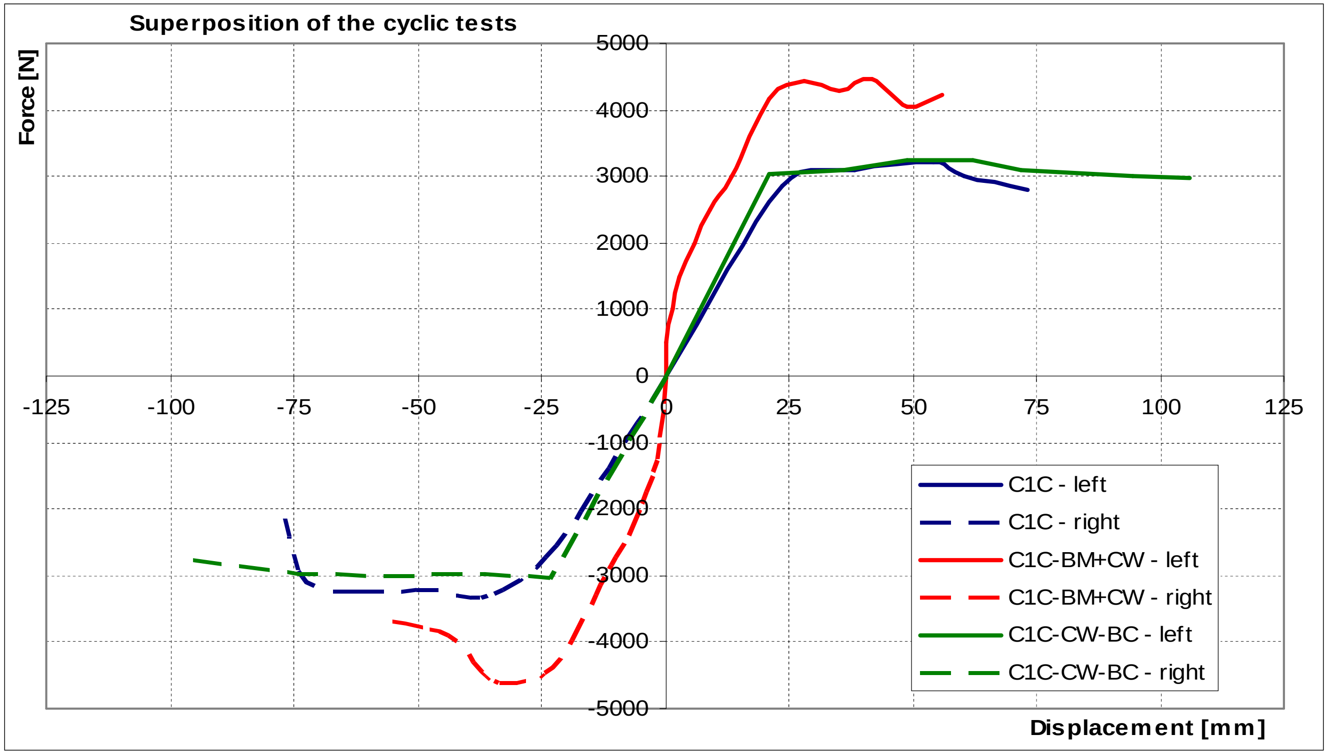
Specimen strengthened using metallic rods anchored into the foundation and a carbon fiber wrap base confinement [C4C-BM+CW] Cyclic testing



Superposition of the individual monotonic tests



Superposition of the individual cyclic tests



Conclusions

RC BEAMS STRENGTHENED WITH FRP SYSTEMS – RESULTS IN DAPPED-ENDS AND INCREASE OF FLEXURAL CAPACITY

- *BY*

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***Cosmin DĂESCU**, PhD Student*
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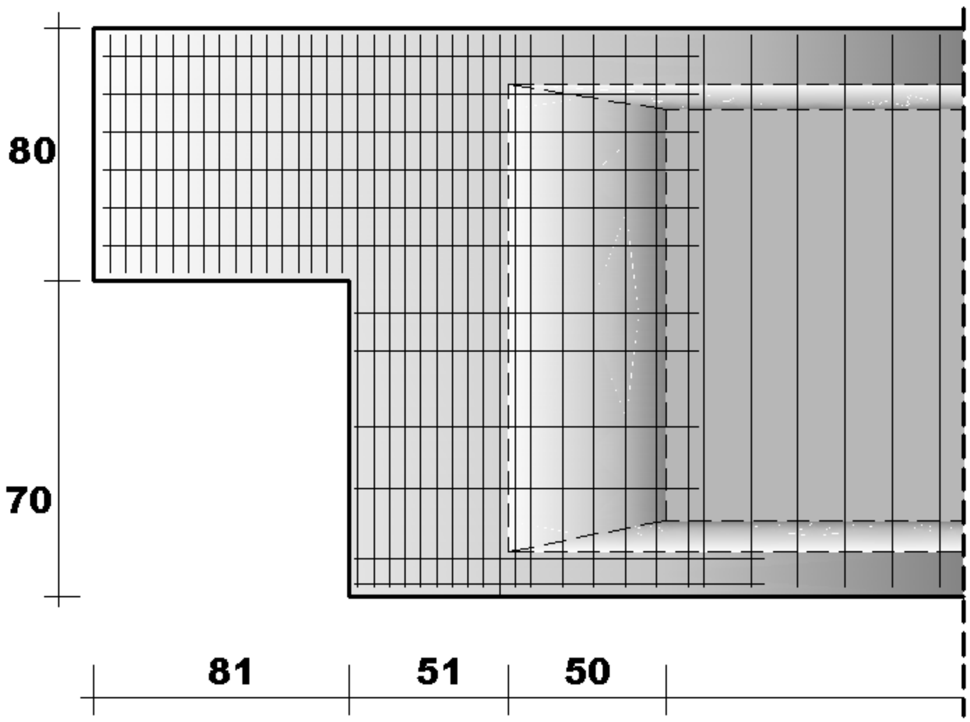
I. DAPPED BEAM ENDS STRENGTHENED WITH FRPS → DIMENSIONING

⇒ PRELIMINARY DIMENSIONING AND DETAILING → ACCORDING TO THE *ROMANIAN CODES* AND VERIFIED WITH *EC2, ACI318, PCI* IN ORDER TO OBTAIN THE BEARING CAPACITY OF 800 KN



BEAM'S HEIGHT → 150 CM
DAPPED ZONE → 80/80 CM
ELEMENT WIDTH → 66 CM

INTRODUCTION

- **Main objectives of the study**

  - the behaviour of dapped beam ends strengthened with FRP composites in various layouts

  - the effectiveness of the systems used

- **Procedure**

1. Theoretical investigation

- simplified design (codes)
- numerical analysis
- strut-and-tie

2. Experimental investigations on full scale beam ends

RESEARCH SIGNIFICANCE

– **For dapped beam end there were identified five potential failure modes (PCI):**

(1) flexure (cantilever bending) and axial tension in the extended end,

(2) direct shear at the junction between the dapped and undapped zone of the member

(3) diagonal tension on the re-entrant corner

(4) diagonal tension in the extended end

(5) diagonal tension in undapped zone

New strengthening techniques

- development of **new strengthening solutions** for this type of elements, with deficient load bearing capacity, caused by design errors, structural or codes modifications, increasing loads or structural damage
- the special advantages offered by the **composites** in structural retrofitting, compared with traditional solutions currently used

- **Previous researches in CFRP strengthening of dapped beam end**

  Huang, Nanni & a., 2000

  Gold, 2001

  Tan, 2001

  Elgwady, 2002

- The present research program has been performed in the following steps:

  - predimensioning and detailing the element,

  - numerical analysis with finite element

  - strut-and-tie method

  - experimental testing on four dapped beam ends strengthened with FRP composites

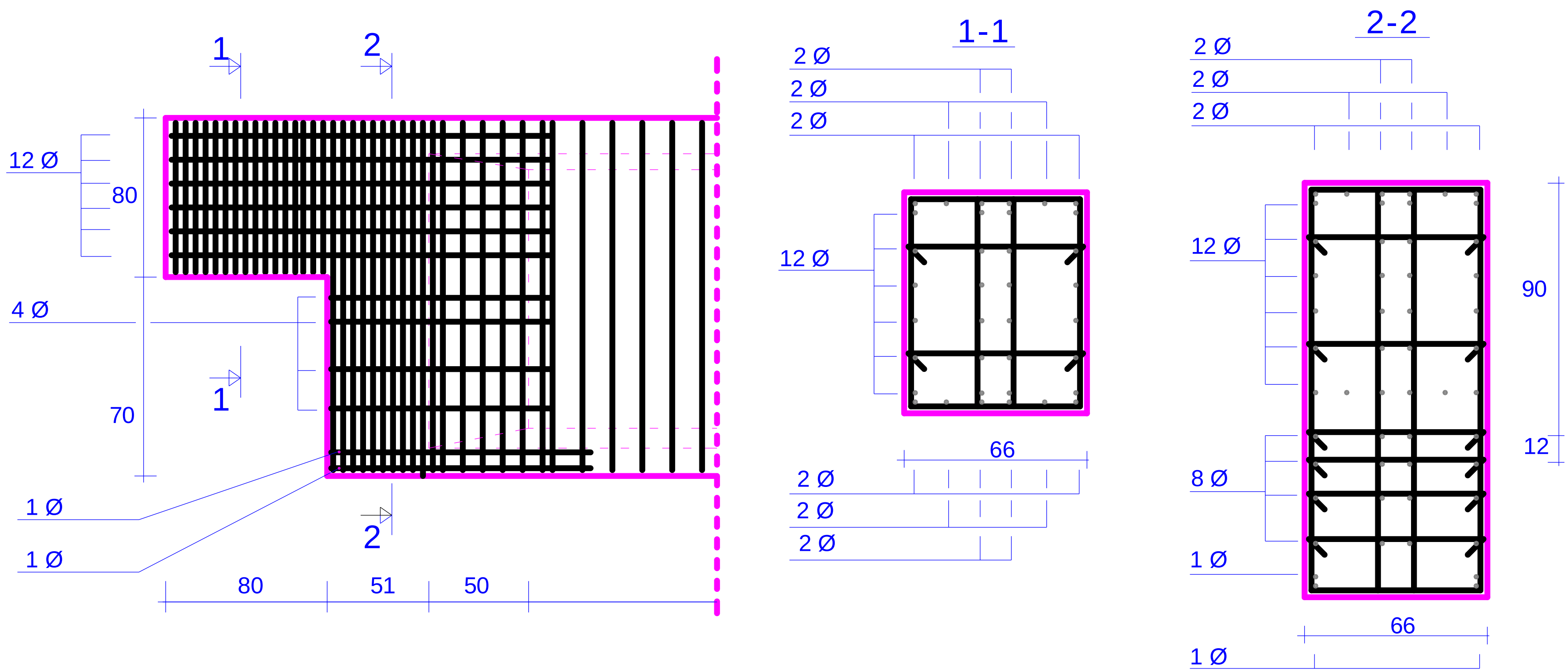
  - interpreting the results and preparing the conclusions

DIMENSIONING AND DETAILING

- Preliminary dimensioning and detailing of the studied dapped beam end were made according to the Romanian codes and verified with those from EC2, ACI318 and PCI Design Handbook, in order to attain the bearing capacity of 80 t (800 kN).

- beam height	= 150 cm,
- dapped zone	= 80/80 cm
- element width	= 66 cm.

ELEMENT DETAILS



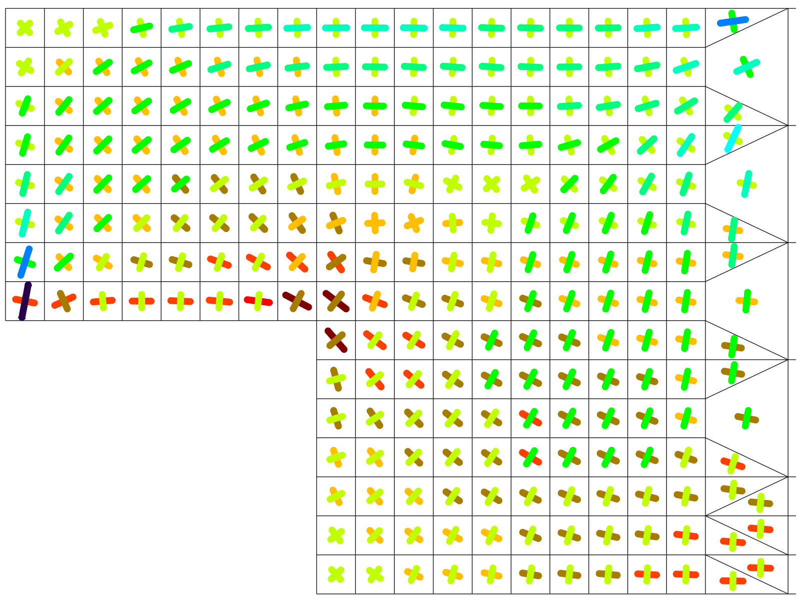
NUMERICAL ANALYSIS

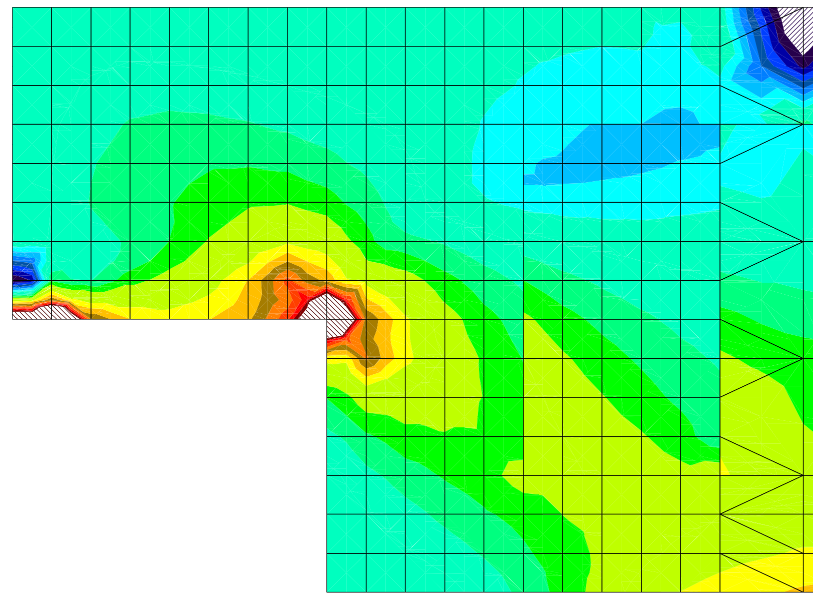
- In the theoretical models, there were used the characteristic strengths of the concrete and the steel reinforcement.

  - elastic analysis (FEM)
  - nonlinear analysis (FEM)
  - Strut-and-Tie (STM)

Elastic Analysis

Direction and distribution of the principal stresses

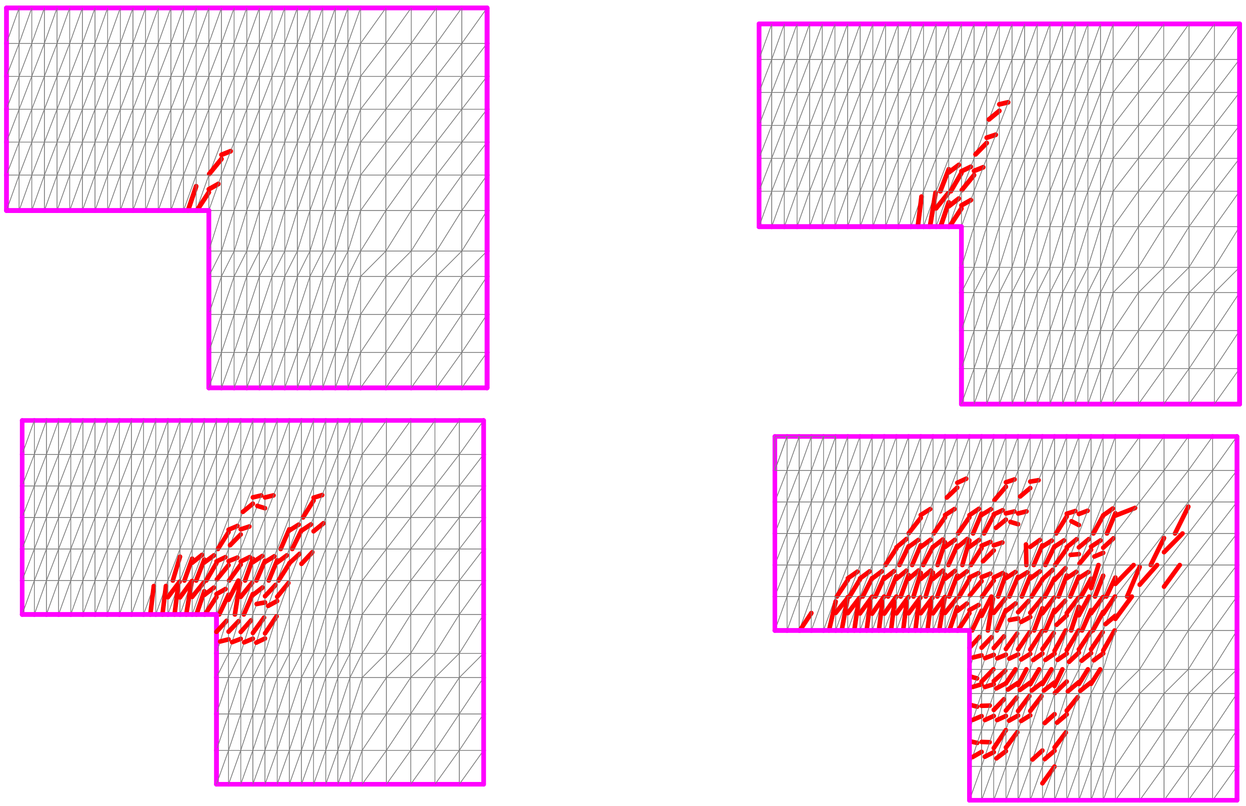




The load level corresponding to the yielding limit in the horizontal reinforcement was 115 t.

Nonlinear Analysis

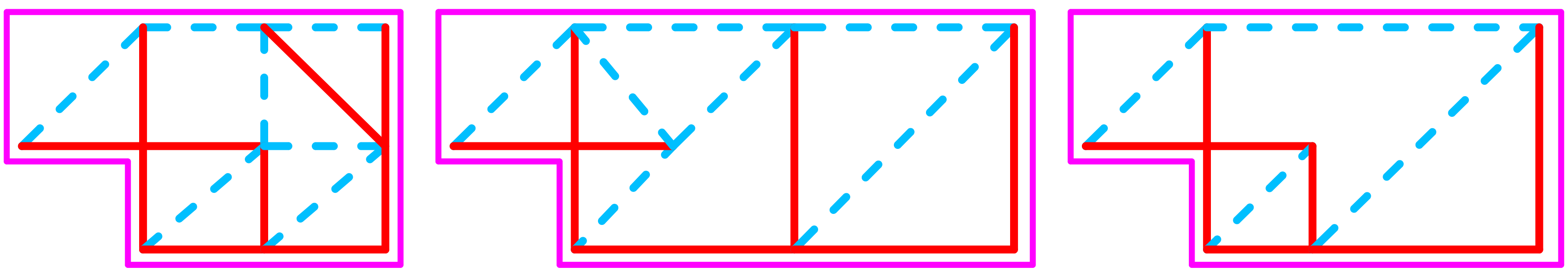
Crack pattern at different load levels



Yielding in the horizontal reinforcement started at the load level of 90 t.

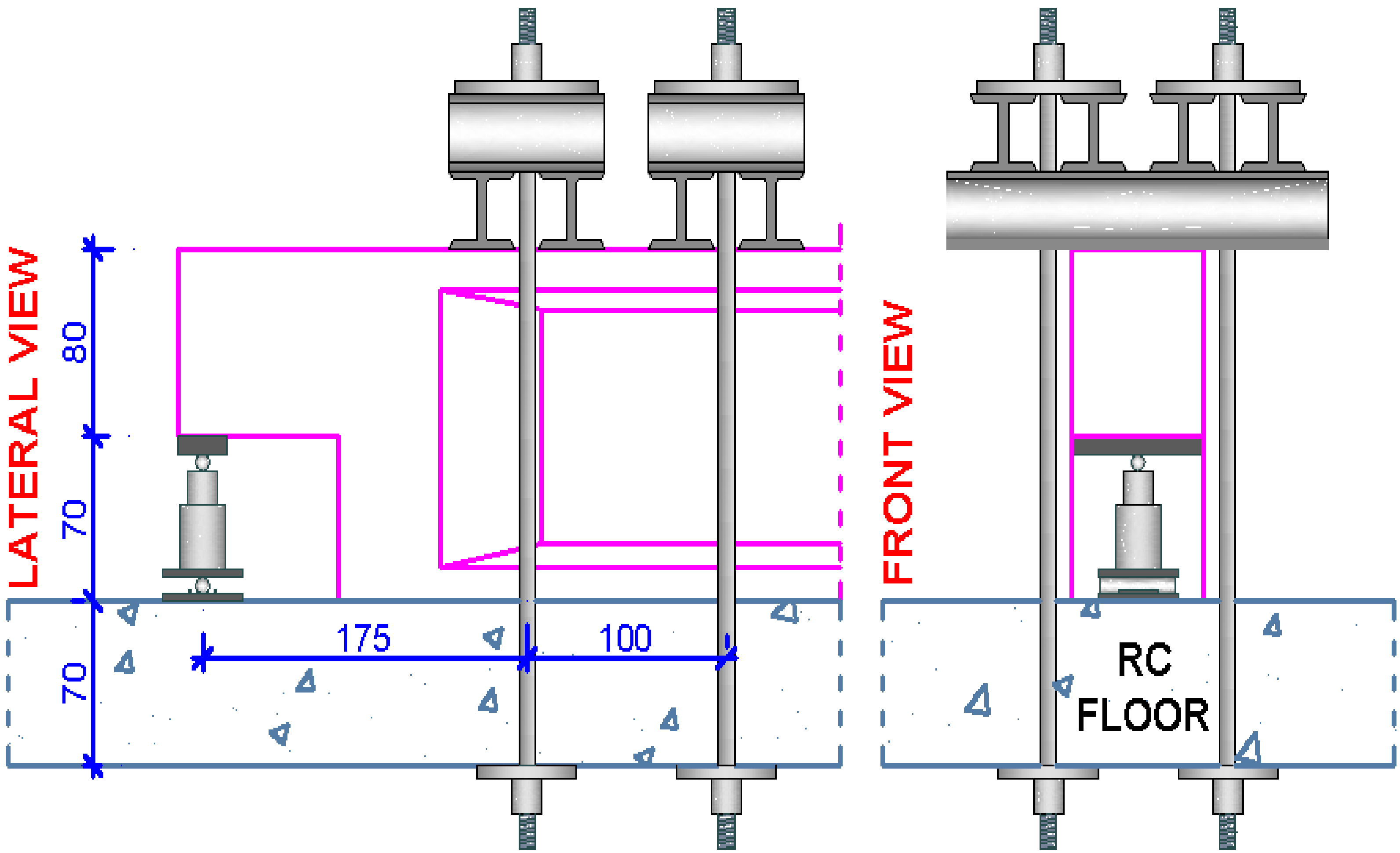
Strut-and-Tie

The analysis was performed to determine the maximum force at which the horizontal bars from the dapped-end are starting to yield.



The maximum force at which the most tensioned reinforcement started to yield was 94 t.

EXPERIMENTAL TESTS



Test results

- The elements showed a similar behaviour as maximum load and deflection
- The design value of the serviceability limit state was 80 t.
- The stress level recorded in the reinforcement was comparable for all the elements;
- The unloading behaviour was similar for all the specimens, very close to the initial starting point;
- It was noted a good similarity between the crack pattern for all the specimens, the general aspect being identical;

Test results – control specimen (C1)

- Specimen C1 was tested close to failure and served as control element.

  - first crack started at an angle of 60°

  - crack pattern was distributed around the re-entrant corner

  - maximum crack width was 3.5 mm

  - maximum displacement was 30 mm

  - peak load was 160 t.

CFRP used characteristics

System	Components	Tensile Strength [N/mm²]	Tensile Modulus [N/mm²]	Strain at Failure [‰]
1 → RC1	SikaWrap 230C Fabric 300 x 0,12 mm	4100	231000	17
	SikaDur 330 Resin	30	3800	-
2 →RC2, RC4	Sika CarboDur S1012 Plate 100 x 1,2 mm	2800	165000	17
	SikaDur 30 Resin	30	12800	-
3 → RC3	SikaWrap 400C HiMod NWFabric300x0,191mm	2600	640000	4
	SikaDur 300 Resin	45	3500	15

Test results–rehabilitated specimen (RC1)

- Specimen RC1 had a linear behaviour up to 160 t

  - the first fibre rupture at 160 t

  - maximum load was 178 t

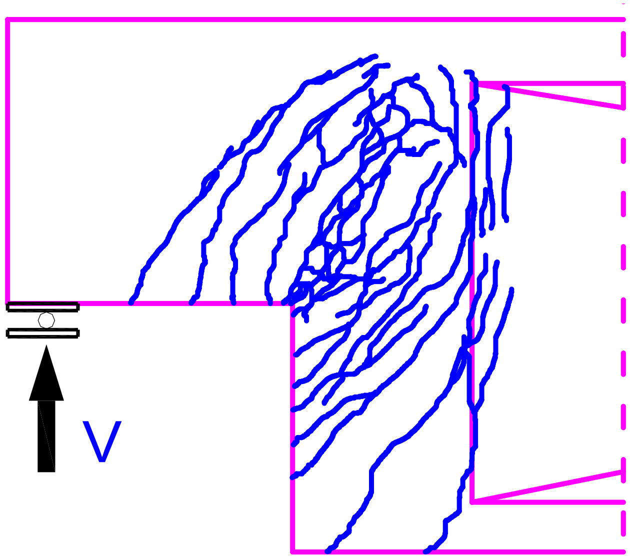
  - yielding until collapse

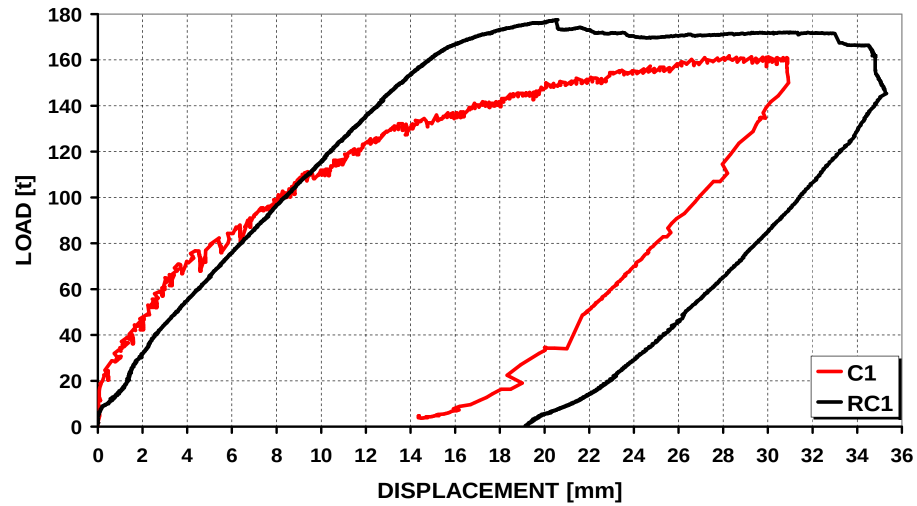
  -failure mechanism: successive breaking of the carbon fibres along main crack and not due to fibre debonding or delamination + concrete crushing in the compressed zone at the maximum load

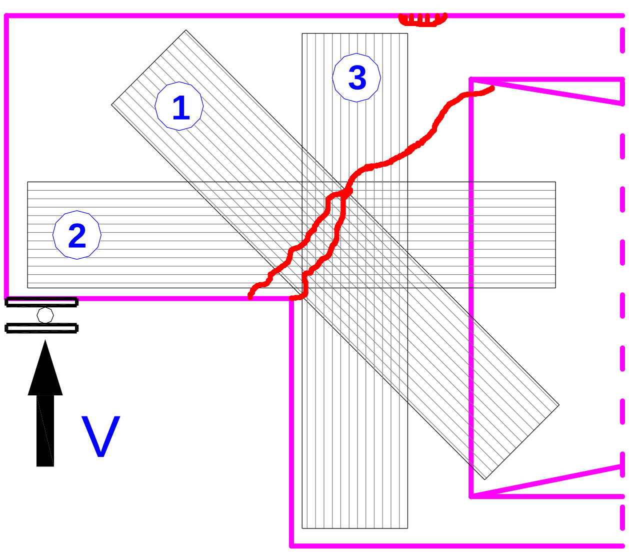
  - maximum displacement was 35 mm

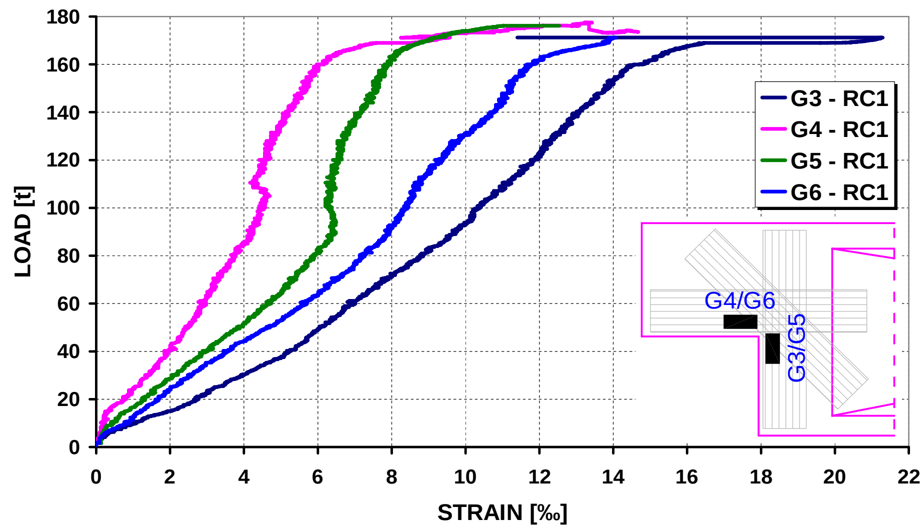
  - strain in fibres reached the maximum values

PART A – STRUCTURAL REHABILITATION USING CFRP









Crack patterns, load-displacement curves and strains in composite (G) of the C1 and RC1 elements

PART A – STRUCTURAL REHABILITATION USING CFRP

Test results–rehabilitated specimen (RC2)

- Specimen C2 was tested up to 80 t.

  - the first crack started at a 42°

  - at 65 t appear four major cracks

  - the maximum displacement was 6 mm

  - strain gages in steel 1.87 ‰ (yielding)

- Specimen RC2 had a linear behaviour up to 130 t

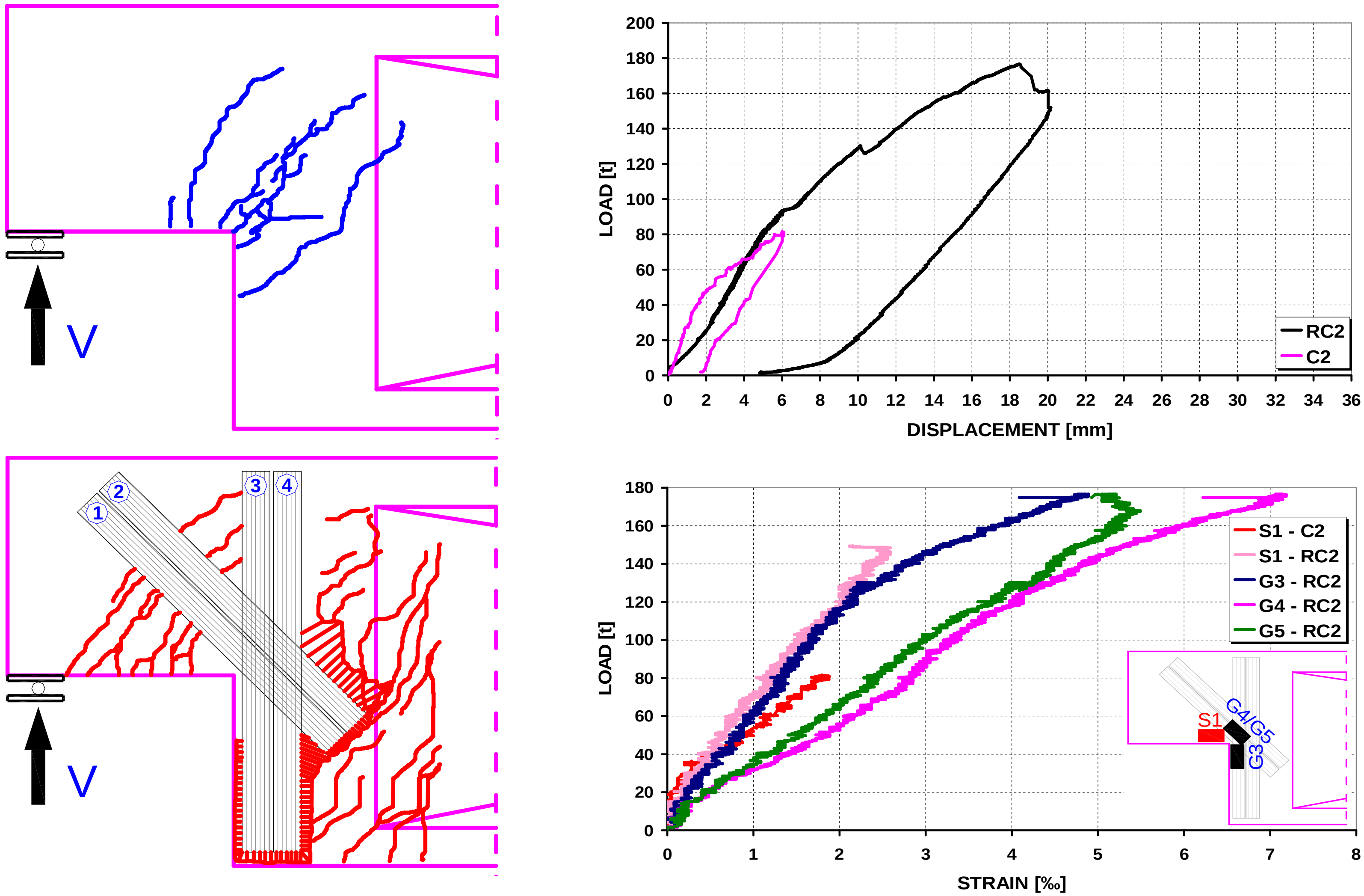
  - at143 t appear the peeling-off of the horizontal plates

  - maximum load at 176 t when also the vertical plates failed through peeling-off; the failure was brittle

  - the maximum strain in steel was 2.59 ‰ at 148 t

  - the maximum strain in composite 7 ‰, which correspond to 41 % of the composite’s ultimate value.

PART A – STRUCTURAL REHABILITATION USING CFRP



Crack patterns, load-displacement curves and strains in steel reinforcement (S) and composite (G) of the C2 and RC2 elements

PART A – STRUCTURAL REHABILITATION USING CFRP

Test results–rehabilitated specimen (RC3)

- Specimen C3 was tested up to 80 t.

  - the first crack started at a 45

  - strain in the steel reinforcement 1.95 ‰

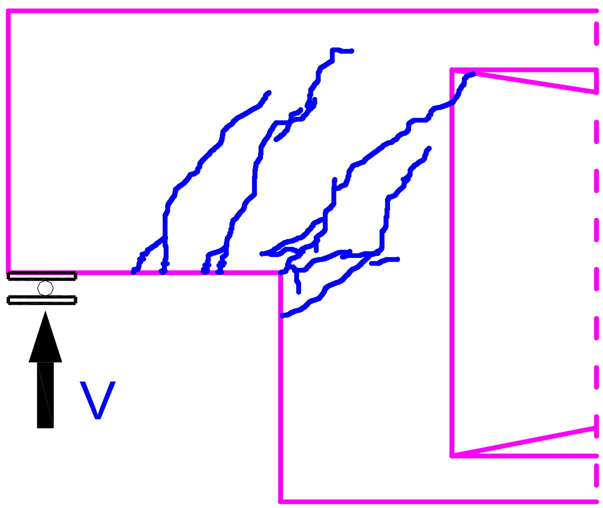
- Specimen RC3 had a linear behaviour up to 90 t

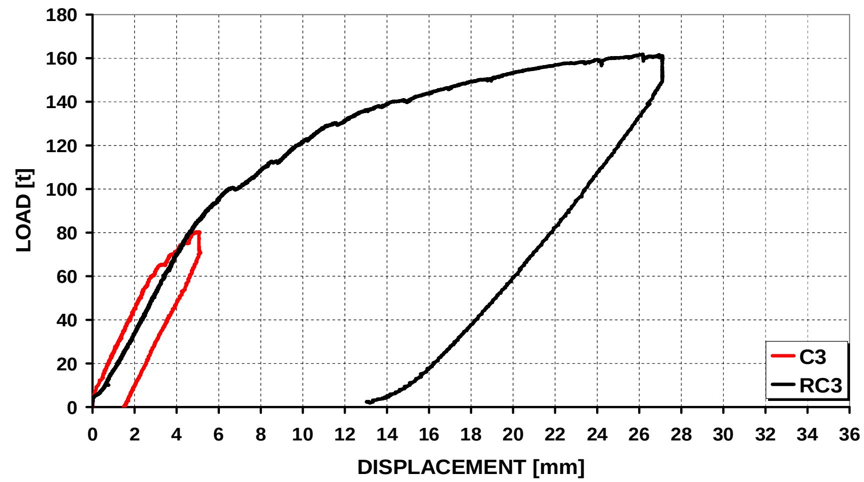
  - aspect is very close to the one of RC1 specimen

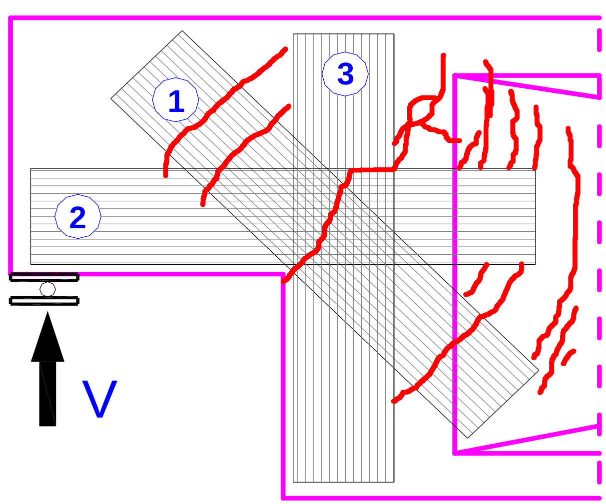
  - the maximum load and remanent displacement were identical with the one from RC1

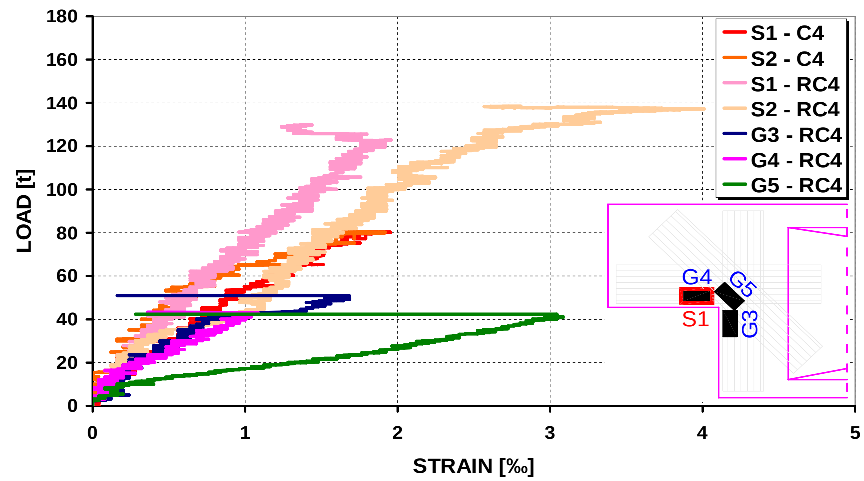
  - the failure was brittle, produced by successive breaking of the carbon fibres along the main crack

PART A – STRUCTURAL REHABILITATION USING CFRP



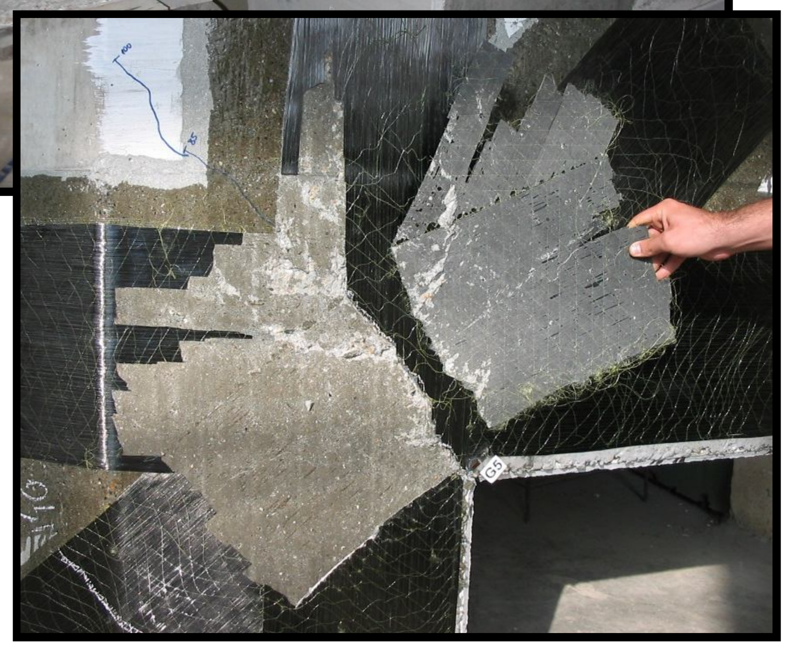






Crack patterns, load-displacement curves and strains in steel reinforcement (S) and composite (G) of the C3 and RC3 elements

PART A – STRUCTURAL REHABILITATION USING CFRP



Test results – rehabilitated specimen (RC4)

- Specimen C4 first crack at 46°

  - after 80t four new major cracks developed

  - strain in steel 1.44 ‰

- Specimen RC4 had a linear behaviour up to 98 t

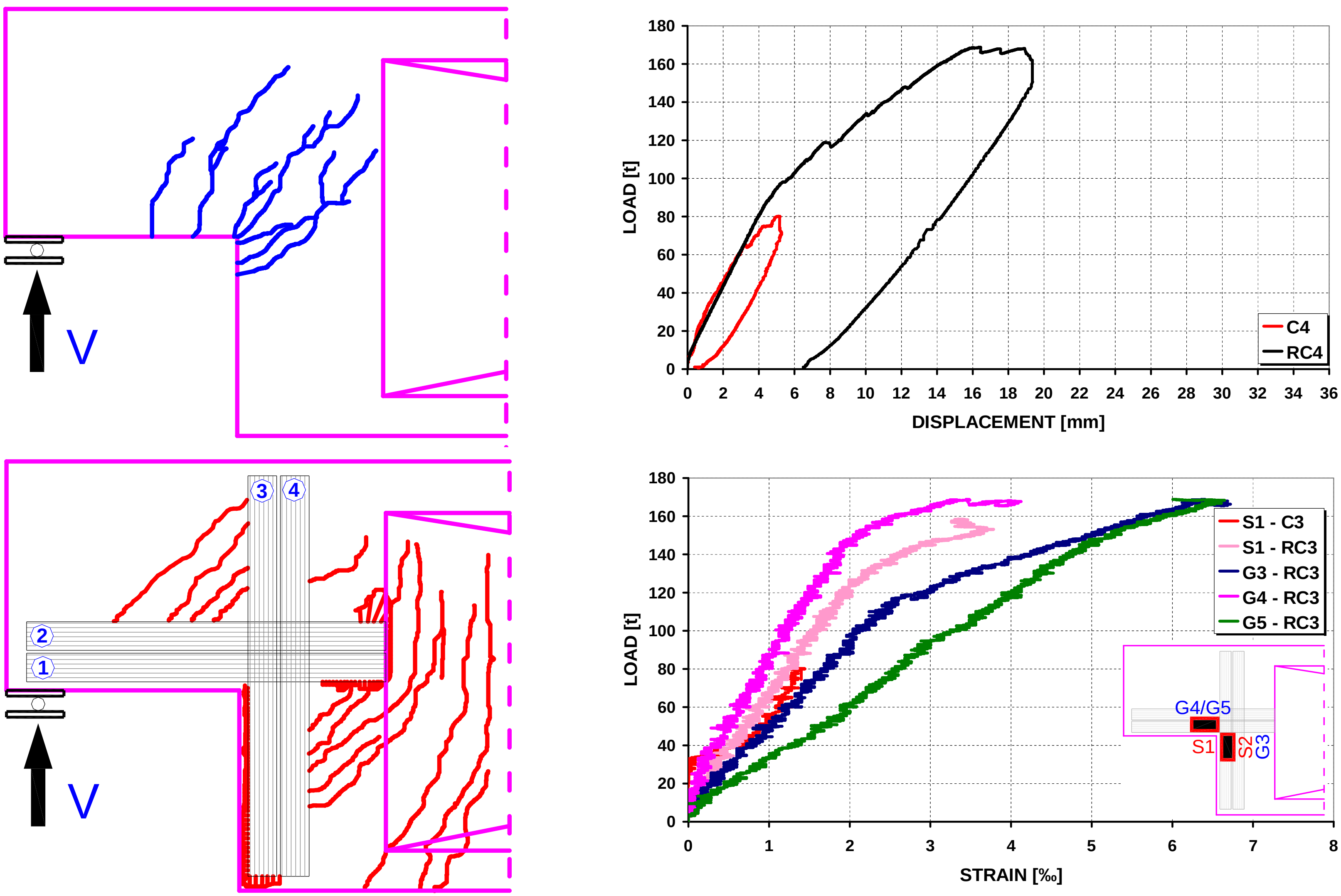
  -at 119 t a crack developed around the horizontal plates.

  - the element failed at 169 t through debonding of vertical plates with an immediate peeling-off of the horizontal plates; the failure was brittle

  - strain in steel 3.78 ‰ at 153 t

  - strain in composite reached 6.72 ‰, which corresponded to 40 % of the composite's ultimate value.

PART A – STRUCTURAL REHABILITATION USING CFRP



Crack patterns, load-displacement curves and strains in steel reinforcement (S) and composite (G) of the C4 and RC4 elements

PART A – STRUCTURAL REHABILITATION USING CFRP

- The used theoretical models approximate with sufficient accuracy the elements' behaviour
- The FRP systems used increase the service load
  (compared with the reference strain in steel reinforcement at 80t)
- The maximum load bearing capacity of the elements increased

Specimen	**+ ΔSL (%)**	**+ ΔUL (%)**	**Δ_{max}(mm)**
RC1	-	11	34
RC2	45	10	18
RC3	25	0	27
RC4	40	6	19

- Elements strengthened with fabrics failed more ductile compared with those retrofitted with plates
- Strengthened elements exhibit a delay in cracking
- The failure occurred by fibre rupture through principal diagonal crack in case of fabric used, and by debonding of the horizontal or inclined plates
- Attention is to be paid for anchorage length in the case of plates

High Performance Materials Used in Retrofitting Structural Masonry Walls

- *BY*

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- *Janos GERGELY, Ph.D., Assoc. Prof., UNC CHARLOTTE, USA*
- *Daniel DAN, Ph.D., Lecturer, "POLITEHNICA" UNIVERSITY OF TIMISOARA*

OBJECTIVES

⇒ STUDY THE BEHAVIOUR OF THE UNREINFORCED CLAY BRICK MASONRY WALLS SUBJECTED TO IN-PLAN SHEAR LOADS STRENGTHENED WITH FRP COMPOSITES ON ONE SIDE

Field of applications:

- SEISMIC RETROFIT OF STRUCTURAL MASONRY WALLS
- IMPROVE THE SHEAR CAPACITY OF THE MASONRY BUILDING BEFORE EARTHQUAKE
- RESTORE THE SHEAR CAPACITY OF THE WALLS

RESEARCH ACTIVITY

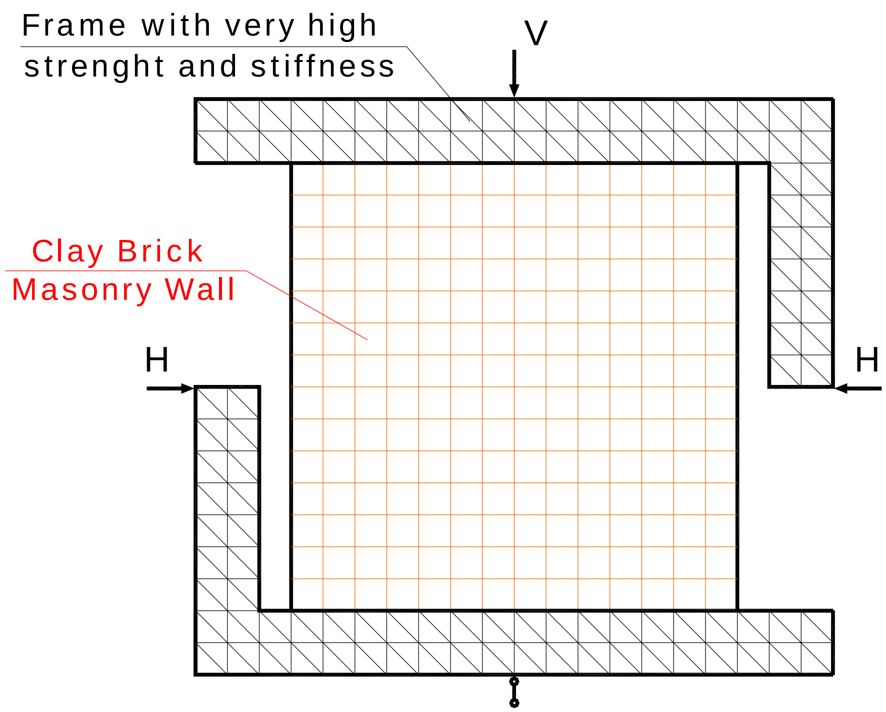
⇒ PERFORMING AN ANALYTICAL STUDY

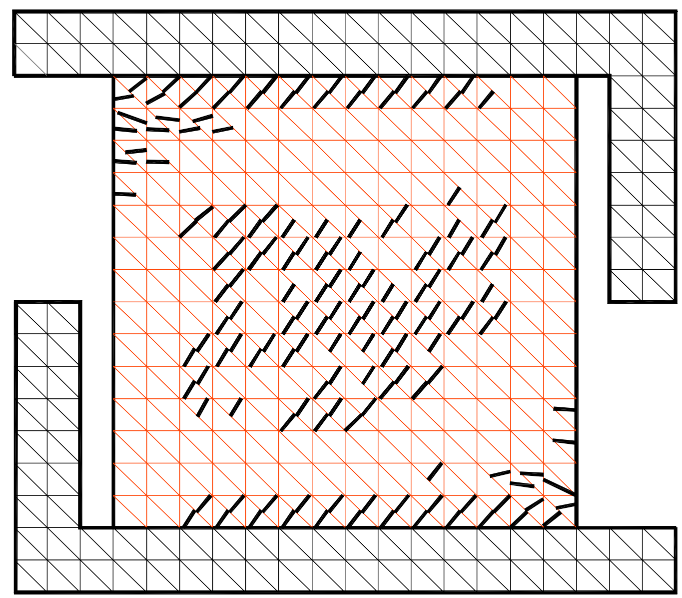
- conceive a device in which the load system creates a pure in-plane shear of the wall, without much influence from the bending moment
- FEA of the walls, modifying: d/h, f_m , E_m , V

⇒ TESTING, RETROFITTING AND RETESTING OF 5 WALLS

FINITE ELEMENT ANALYSIS

THEORETICAL MODEL

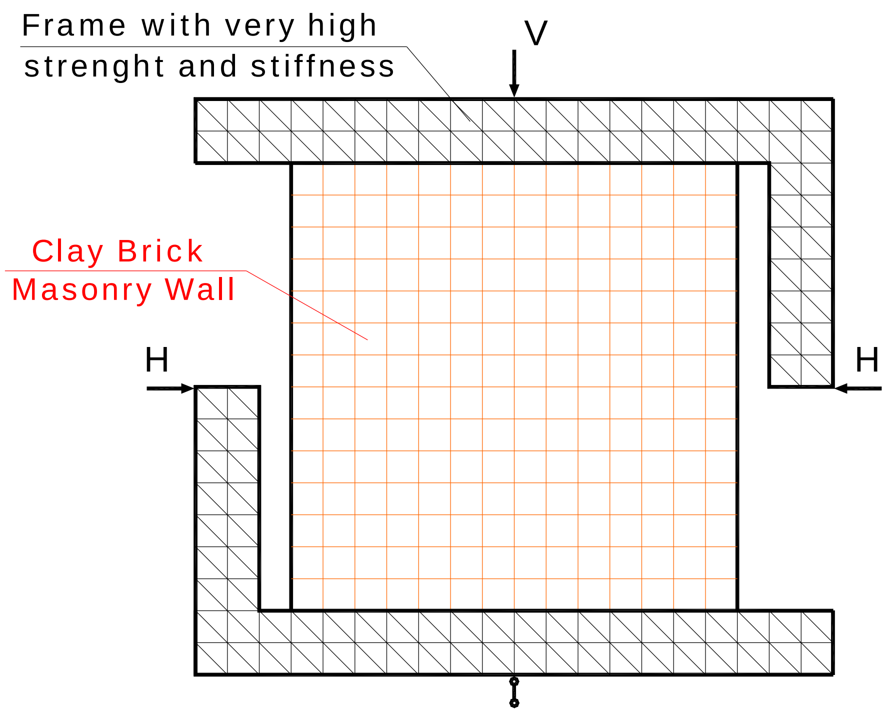


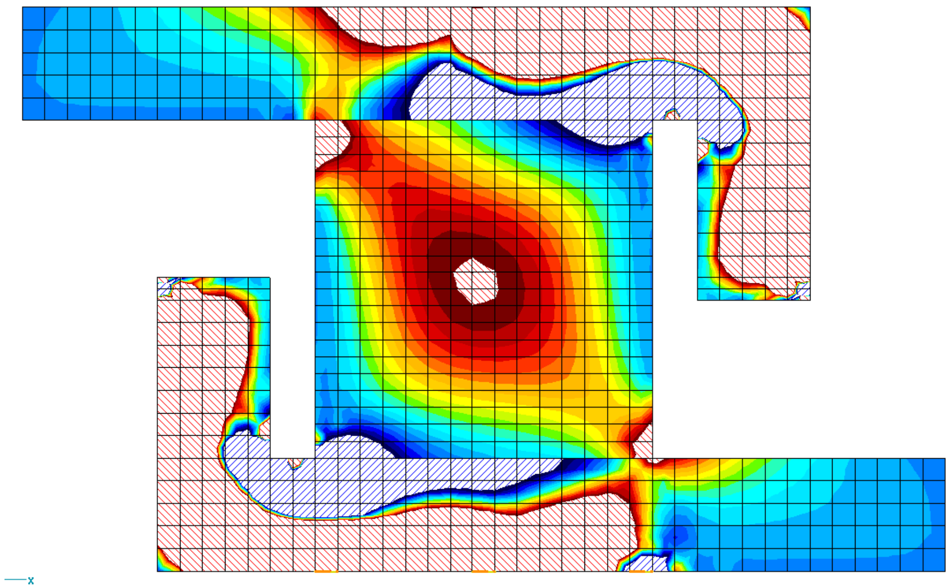


BIOGRAF

FINITE ELEMENT ANALYSIS

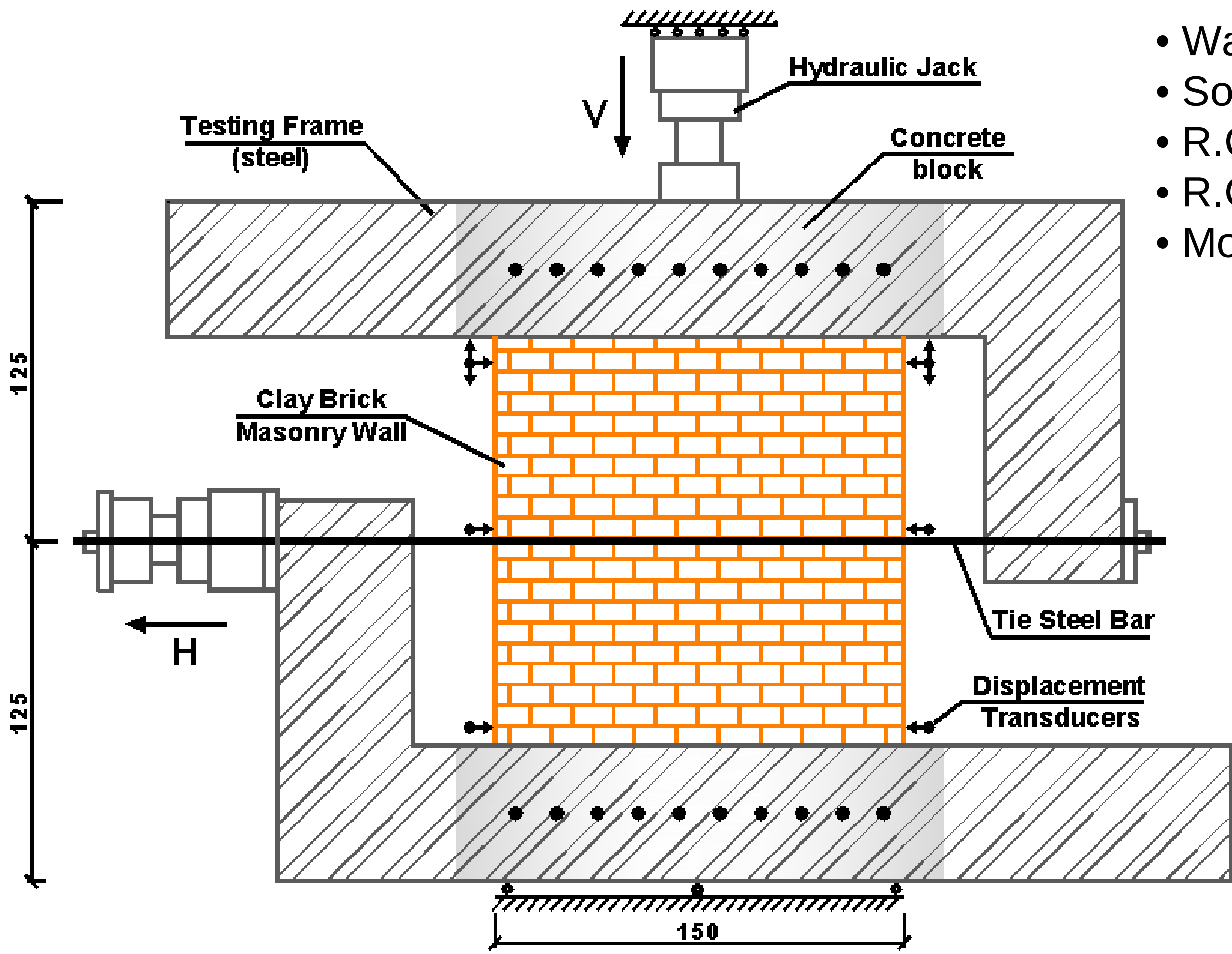
THEORETICAL MODEL





AXIS VM

SPECIMENS TEST SET-UP



- Wall dimension : 150x150 cm
- Solid clay bricks units
- R.C. contour beam: 50x150x25 cm
- R.C. base: 50x150x25 cm
- Mortar strength > M10 (N/mm^2)

WALLS RETROFITTED WITH SWM

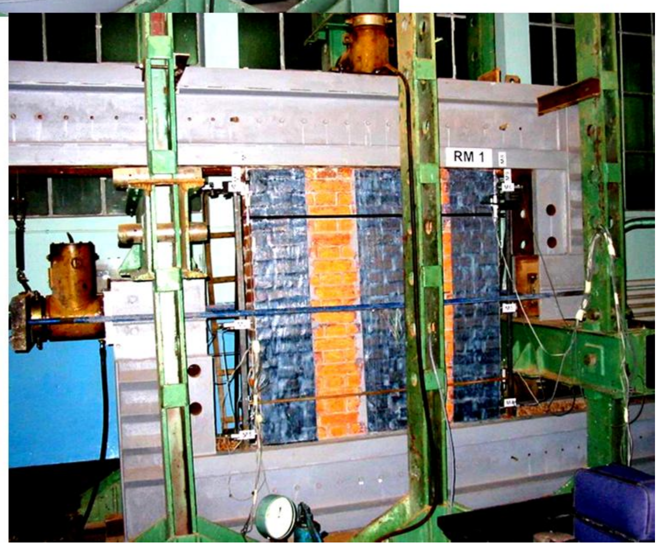
- SEEMS TO BE A **PROMISING SOLUTION**
- WAS STUDIED THE **CONTRIBUTION** OF THIS TECHNIQUE

THE *UM* SPECIMEN WAS **STRENGTHENED BEFORE** TESTING WITH SWM

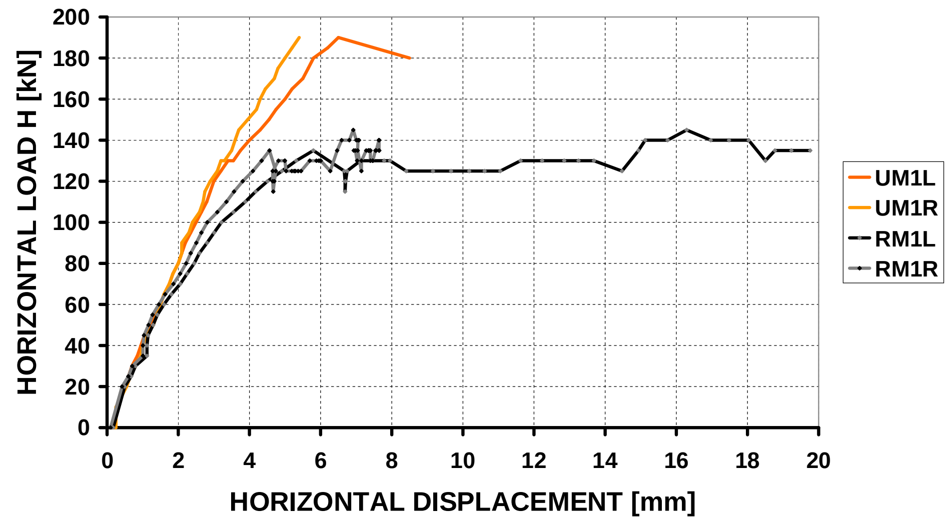
→ **STAINLESS STEEL BIDIRECTIONAL FABRIC** (WIRE MESH) WITH 0.4 mm DIAMETER AND 1.0 mm SPACING, APPLIED WITH AN EPOXY BASED MORTAR.

→ STRENGTHENING PROCEDURE **SIMILAR AS THE USED FOR FRP** SYSTEMS

TEST RESULTS



ε_{frp}=0.50%



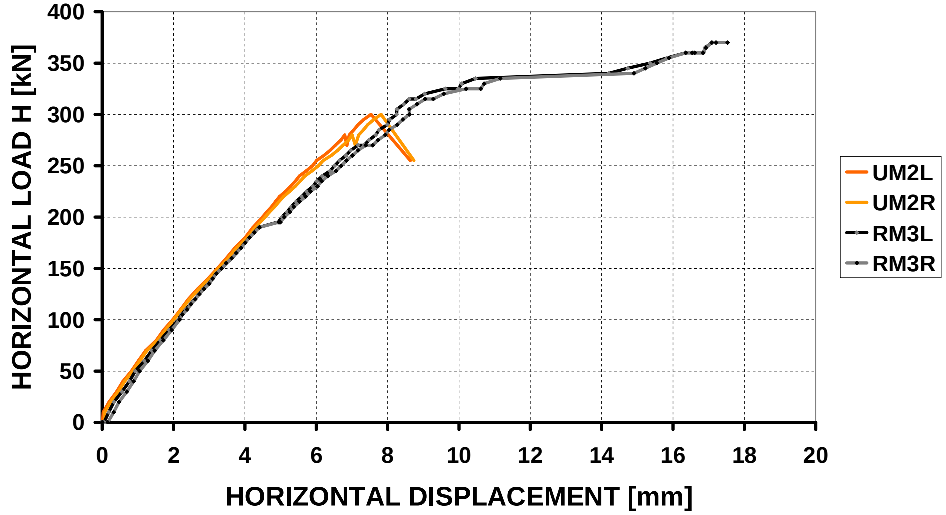
Differences in capacity : - 23%

Difference in displacement : + 131%

<u>FAILURE</u>

→ EXTENSIVE OPENING OF THE EXISTING CRACK FOLLOWED BY COMPOSITE DEBONDING AT THE CRACKS

TEST RESULTS



Differences in capacity : + 23%

Difference in displacement : +101%

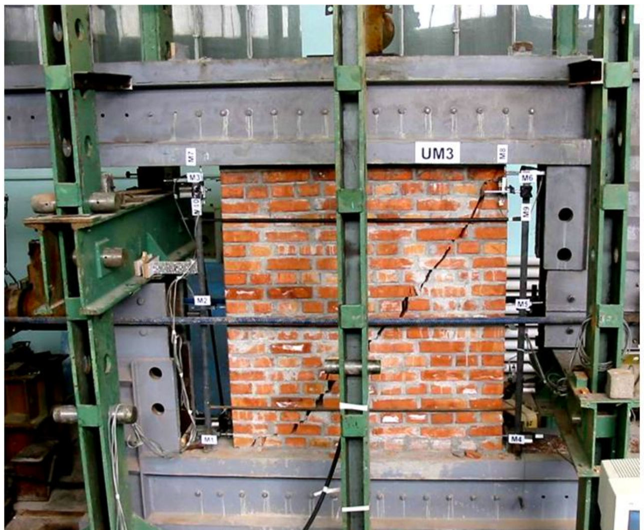
FAILURE

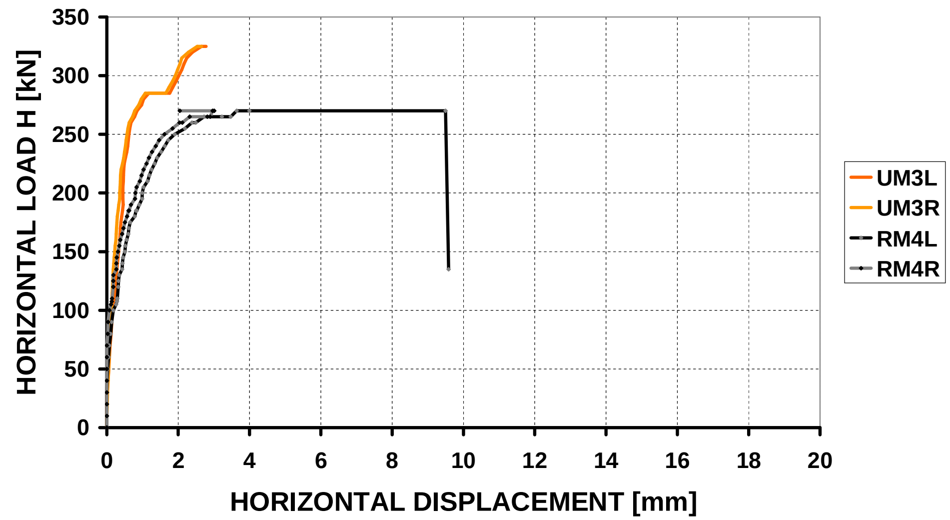
→ DEVELOPMENT OF A NEW CRACK AND THROUGH ITS EXTENSIVE OPENING

→ THE COMPOSITE WAS DEBONDED JUST IN THE CRACK ZONE

ε_{frp}=0.15%

TEST RESULTS





Differences in capacity : - 16%

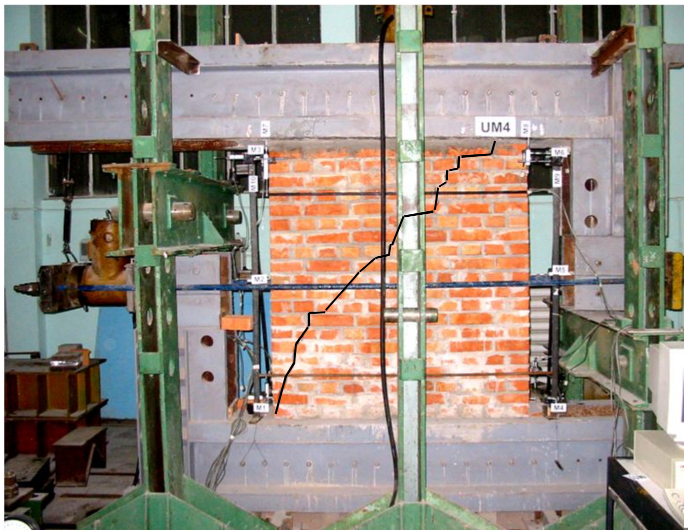
Difference in displacement : +239%

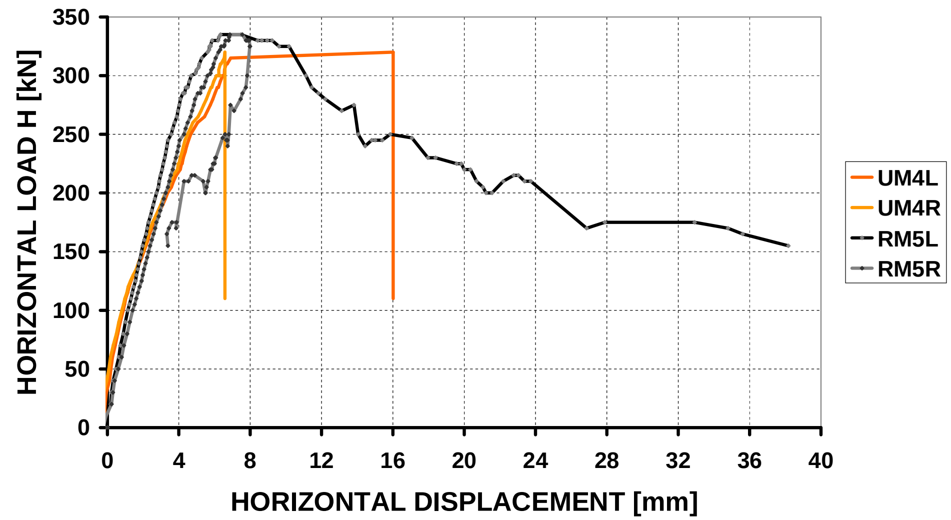
FAILURE

→ EXTENSIVE OPENING OF THE EXISTING CRACK FOLLOWED BY COMPOSITE DEBONDING JUST IN THE CRACK ZONE

ε_{frp}=0.12%

TEST RESULTS





Differences in capacity : + 4%

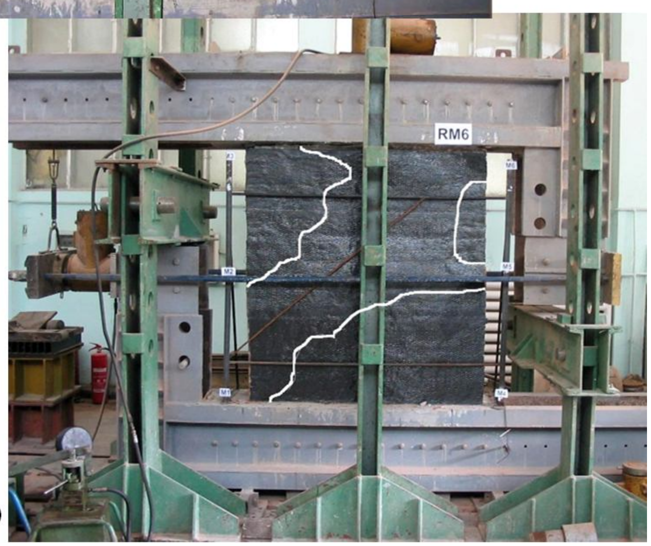
Difference in displacement : +138%

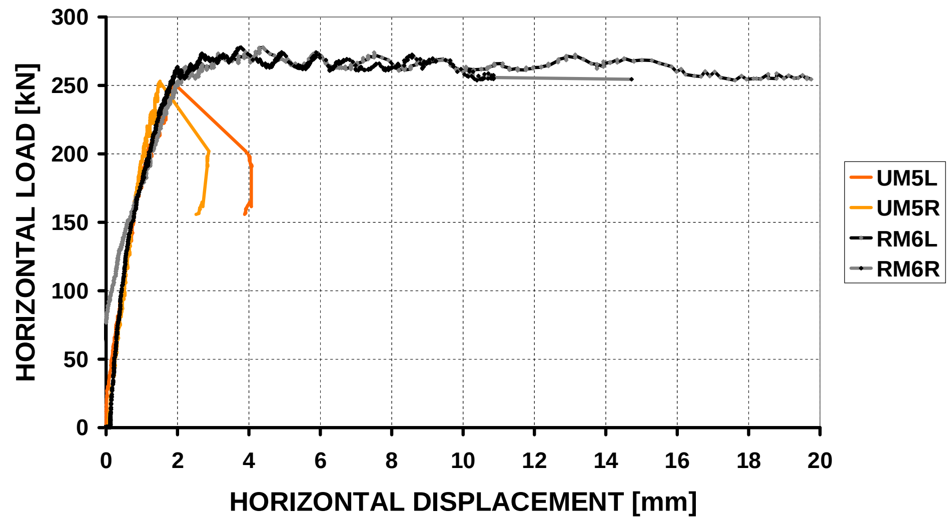
FAILURE

→ EXTENSIVE OPENING OF THE EXISTING CRACK FOLLOWED BY COMPOSITE DEBONDING AT THE CRACKS

ε_{frp}=1.78%

TEST RESULTS





Differences in capacity : + 10%

Difference in displacement : +475%

FAILURE

→ FORMING OF MANY NEW CRACKS AND THROUGH THE EXTENSIVE OPENING OF THE EXISTING ONE

→ THE COMPOSITE DEBONDED ON LARGE AREAS, NEAR THE CRACK ZONE

ε_{frp}=0.18%

CONCLUSIONS

- The correction and injection mortars had an important role in restoring the load bearing capacity

- A considerable capacity increase was observed for the precracked shear walls retrofitted with FRP composites
(practically, the load bearing capacity of the cracked walls was negligible)

- The failure of the retrofitted walls was caused by extensive cracking followed by FRP debonding and not due to tensile or shear failure of the FRP (vertical applied composites debonded just in the vicinity of the major crack, while those applied horizontally debonded in large areas)

- The maximum horizontal displacements increased at least twice compared with the displacements of the reference specimens that demonstrated the increase of the ductility

- The most advantageous system → glass fabric applied dry application process

APPLICATION OF THE RESULTS

(courtesy of Sika Romania)

• In 2002, after an earthquake, two residential buildings with masonry structure, built in 60’, situated in town Moldova Nouă, Romania, were damaged

ON-GOING PROJECT

FP6 - PROHITECH

EARTHQUAKE PROTECTION OF HISTORICAL BUILDINGS BY REVERSIBLE MIXED TECHNOLOGIES

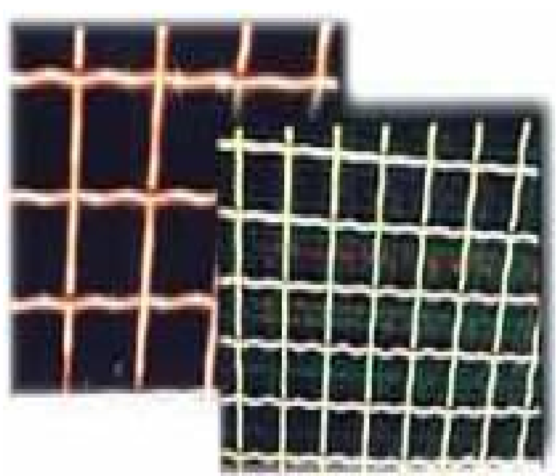
→ Retrofitting masonry walls with Steel Wire Mesh (SWM)

SWM: - bidirectional

- application with epoxy resin (similar with FRP)

a) zinc coated steel wires $R_u \approx 515$ N/mm^2

b) stainless steel $R_u \approx 700$ N/mm^2



SIMPLIFIED MODELS FOR SHEAR RETROFIT OF MASONRY WALLS USING FRP (Glass, Carbon, Steel) AND STEEL SHEATHED TECHNOLOGY

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PURPOSE

- establish a simplified design formula

PROCEDURE

- study the behavior of the structural elements →test
- push over test of experimental specimens
- establish the collapse mechanism
- propose the design formula
- calibrate the parameters in the design formula for the retrofitted masonry using:
  - **experimental tests**
  - **numerical analysis**

MAIN CONCLUSION FROM TESTS:

- retrofitted masonry: **composite element**
- behavior: **intense cracking process** of the composite system
- first phase: formation of smeared crack
- second phase: develop a general diagonal crack
- third phase: damage of the retrofitting material or system
- collapse: **general diagonal crack when the composite system stops functioning**

IMPORTANT NOTICE:

- **full displacement compatibility** between the retrofitting material or system and the masonry up to collapse
- **different damage process** of the retrofitting system:
  - **debonding** of the **GFRP** or **CFRP**
  - **yielding** of the **SFRP**
  - **plastic bearing** of **S**teel **S**heet accompanied by vanishing the composite character (equivalent to debonding)

SMEARED CRACKS

DIAGONAL CRACK

DEBONDING GFRP

DEBONDING CFRP

YIELDING SFRP

PART A – STRUCTURAL REHABILITATION USING CFRP

PLASTIC BEARING

PRELIMINARY DESIGN FORMULA

$$Q_{cap} = f_{rm}.t.l\left(1+\sqrt{1+c\,\frac{\sigma_o\cdot\varphi}{f_{rm}}}\right)$$

$$\varphi \quad = \quad 0 \quad \div \quad 1$$

$$c \quad = \quad 0.2 \quad - \quad 0.8$$

PARAMETERS TO BE EVALUATED

$$f_{rm}\ ,\ \varphi\ ,\ c$$

CALCULATION EXAMPLE

f_{rm} = 0.5 N/mm^2

t = 250 mm

l = 2151 mm

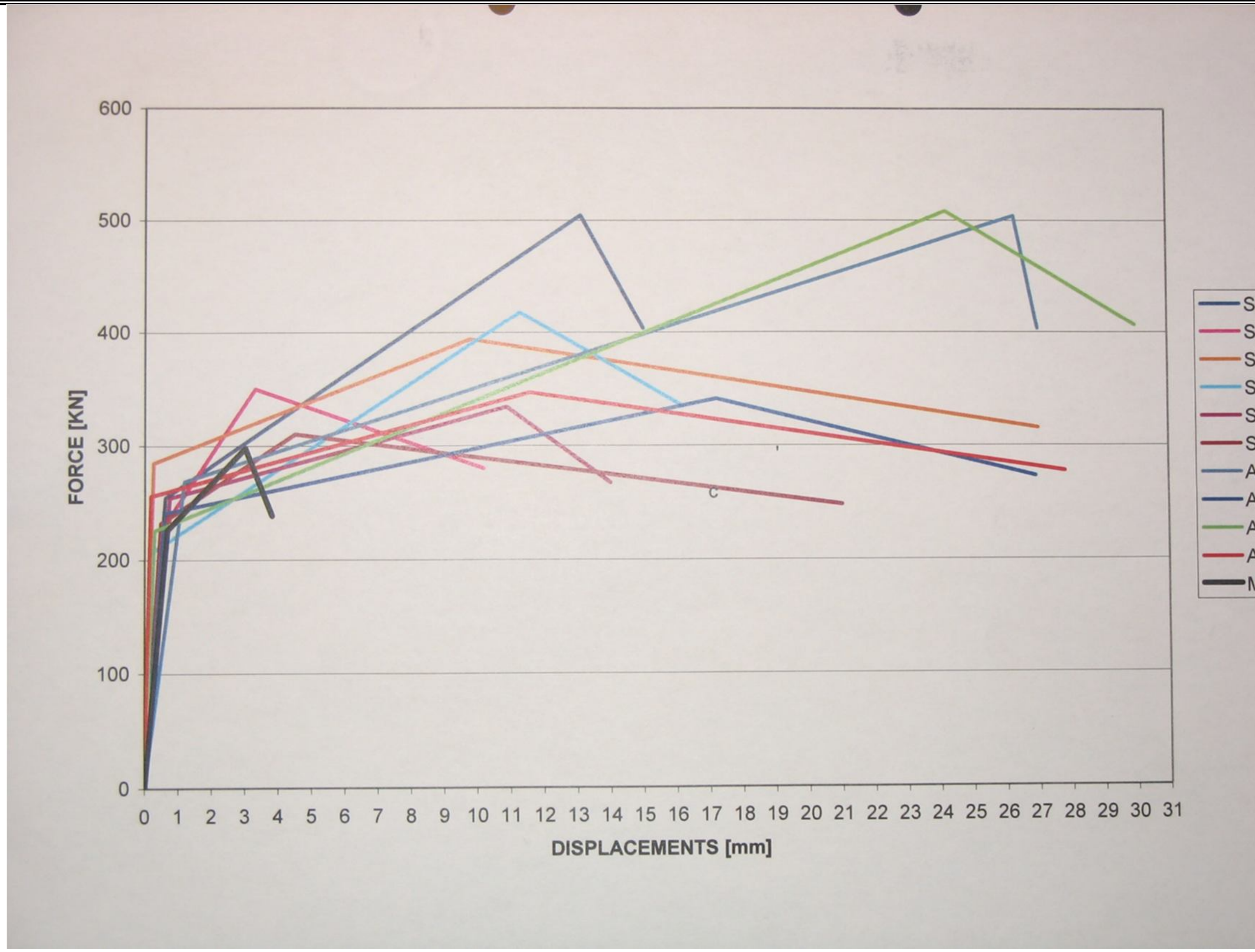
σ_o = 0.5 N/mm^2

c = 0.8,

Φ = 1.0

Q = 360 kN

EXPERIMENTAL REZULTS



CONCLUSIONS:

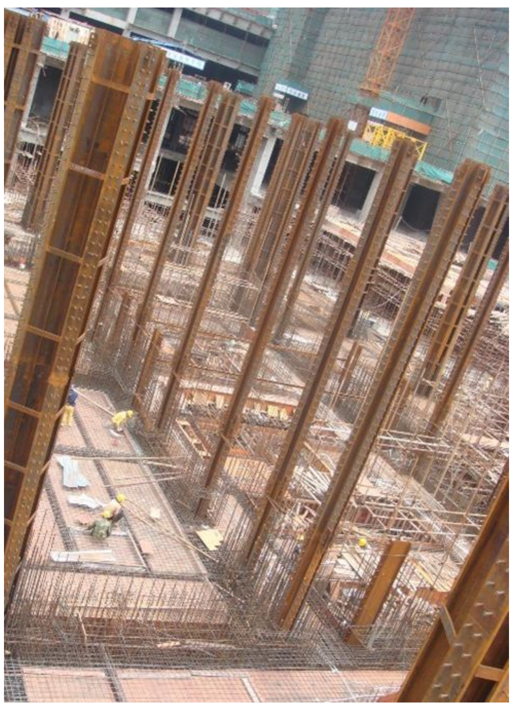
-THE PRPOSED MODEL SEEMS TO BE SUITABLE FOR THE PRACTICAL DESIGN OF RETROFITT MASONRY WALLS

-THE NUMERICAL ANALYSIS IS PERFORMED USING BIOGRAF SOFTWARE

-THE PROPOSED DESIGN FORMULA AND THE SOFTWARE CAN BE APLLIED FOR R.C. STRUCTURAL WALLS

Introduction

COMPOSITE STEEL-CONCRETE SHEAR WALLS ARE REINFORCED CONCRETE WALLS OBTAINED BY REPLACING THE VERTICAL REINFORCEMENTS FROM THE EDGES BY STEEL ENCASED PROFILES



Examples of the solution in practice

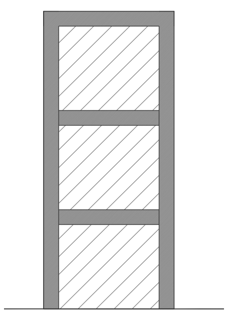
Experimental specimens

Introduction

COMPOSITE STEEL-CONCRETE SHEAR WALLS ARE OFTEN USED AS LATERAL LOAD RESISTING SYSTEMS FOR HIGH RISE BUILDINGS PLACED IN SEISMIC AREAS

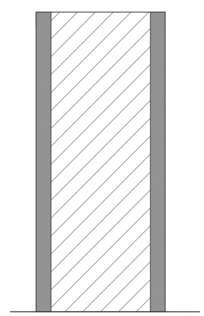
Composite walls structural systems $\frac{\alpha_u}{\alpha_1} \approx 1.1$

Type 1
Steel or composite moment frame with connected concrete infill panels



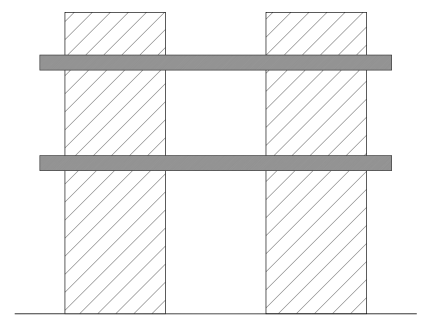
Ductility class	
H	M
$4\frac{\alpha_u}{\alpha_1}$	$3\frac{\alpha_u}{\alpha_1}$

Type 2
Concrete walls reinforced by connected encased vertical steel sections



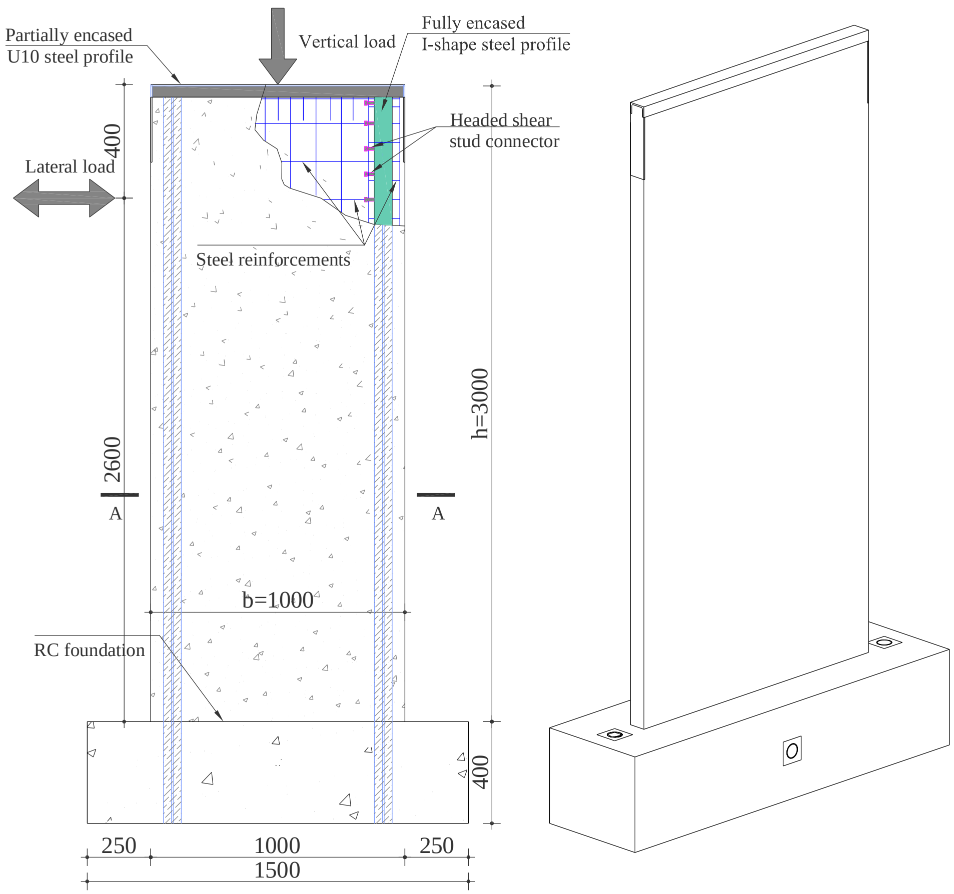
Ductility class	
H	M
$4\frac{\alpha_u}{\alpha_1}$	$3\frac{\alpha_u}{\alpha_1}$

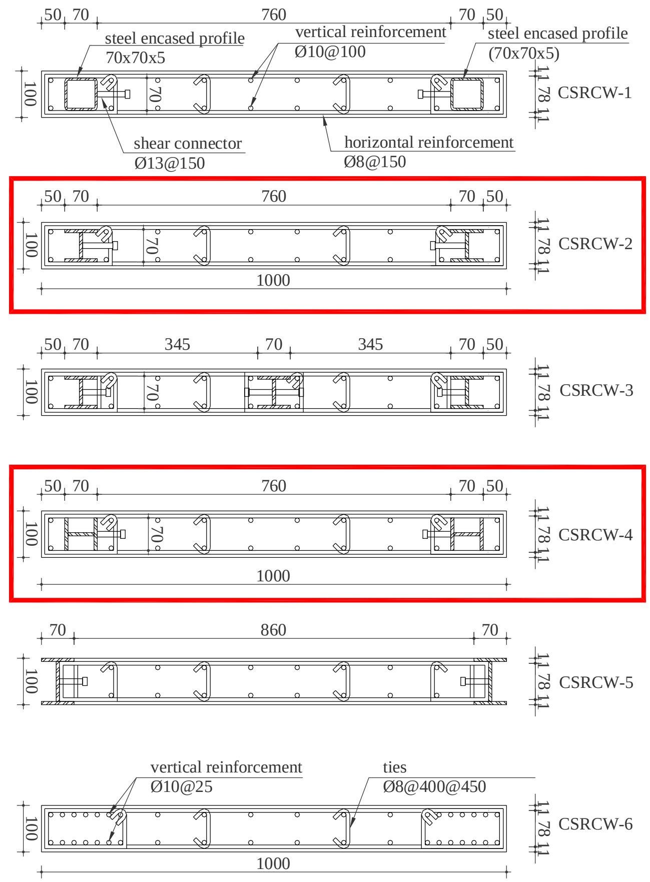
Type3
Composite or concrete walls coupled by steel or composite beams



Ductility class	
H	M
$4.5\frac{\alpha_u}{\alpha_1}$	$3\frac{\alpha_u}{\alpha_1}$

Experimental program





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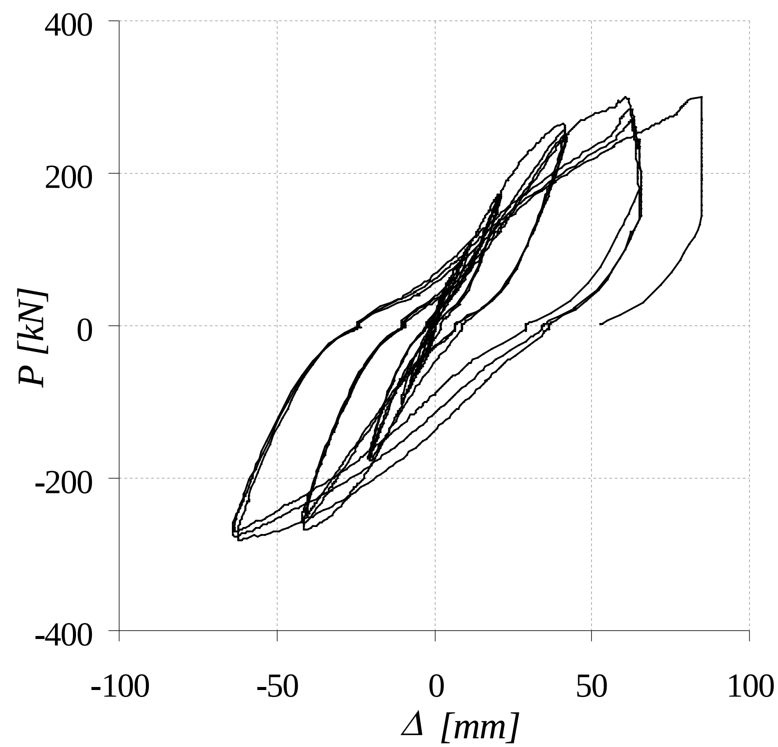
Experimental program

- EXPERIMENTAL RESULTS ON **CSRCW2** SPECIMEN



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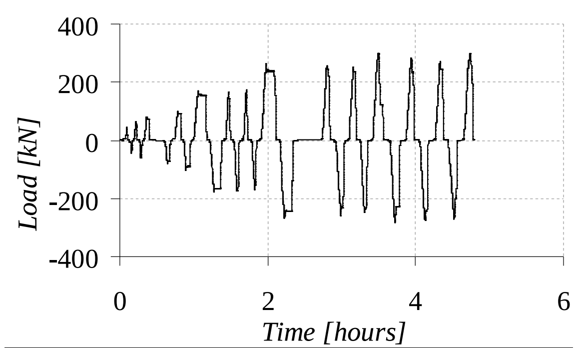

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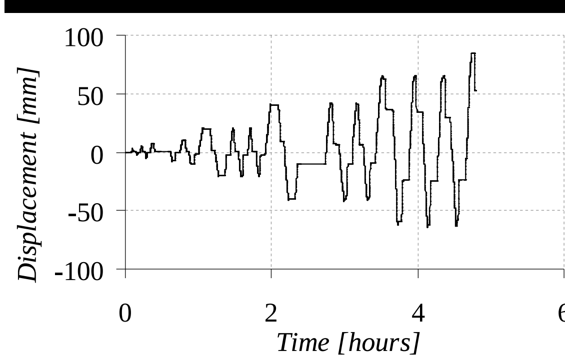

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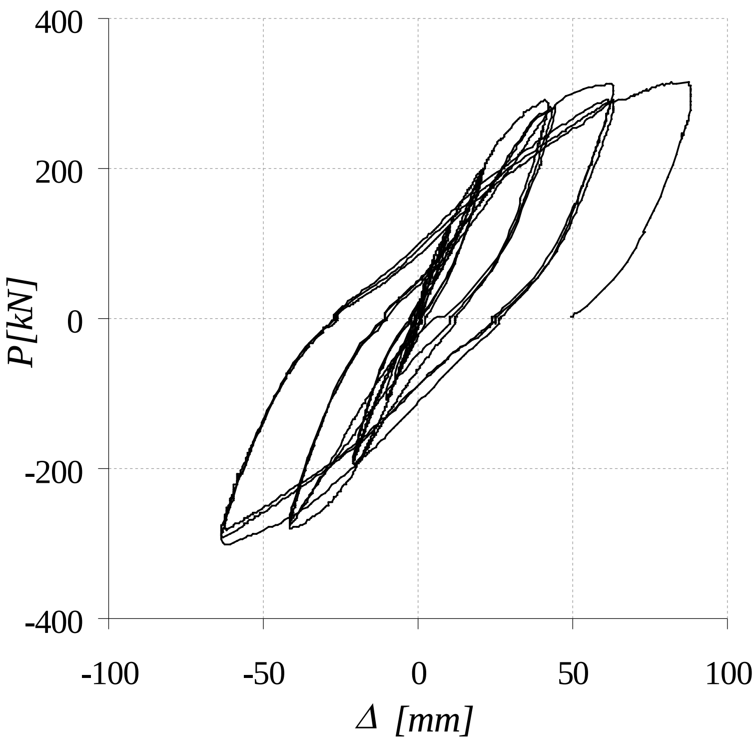
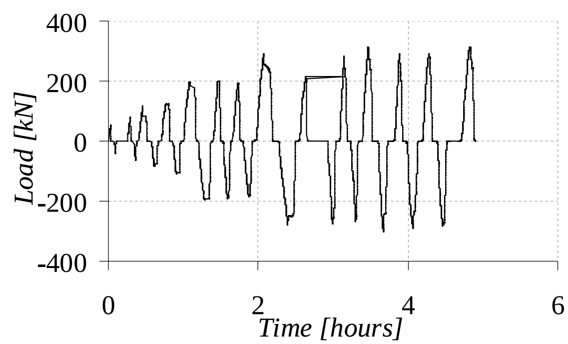
Experimental program

- EXPERIMENTAL RESULTS ON **CSRCW4** SPECIMEN





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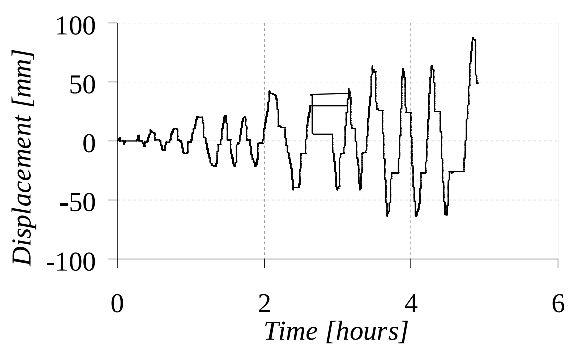

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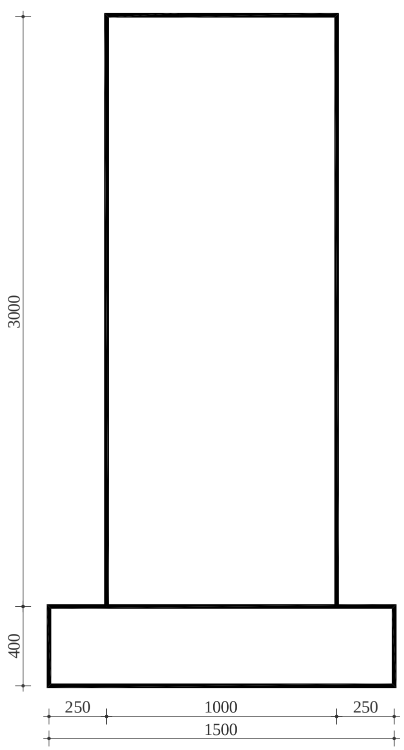
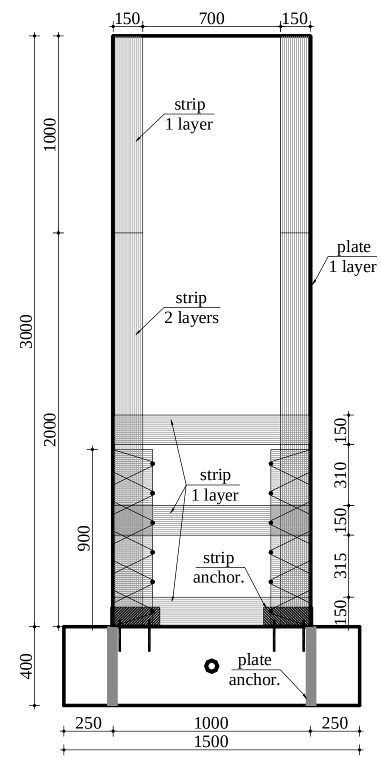
Experimental program

- Experimental investigation on the overall behavior of the damaged and retrofitted composite steel-concrete walls
- Restore the load bearing capacity caused by damages
- Investigate the effectiveness of retrofitting by using externally bonded CFRP composites





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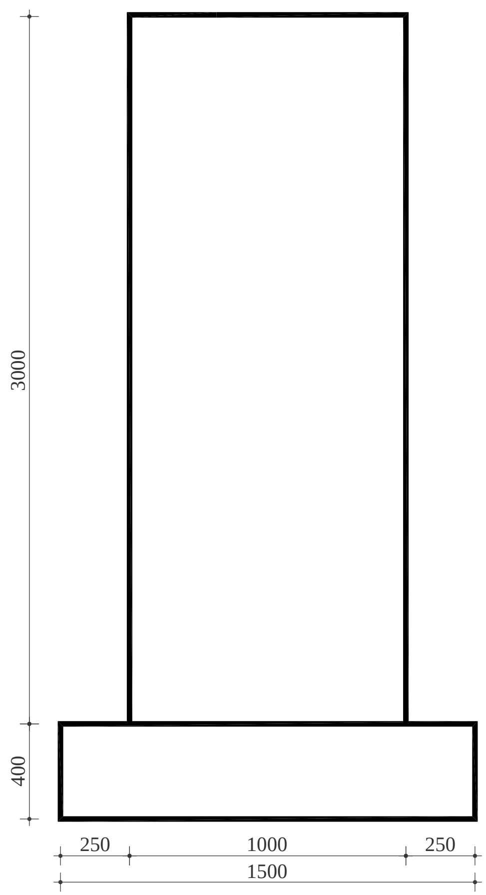
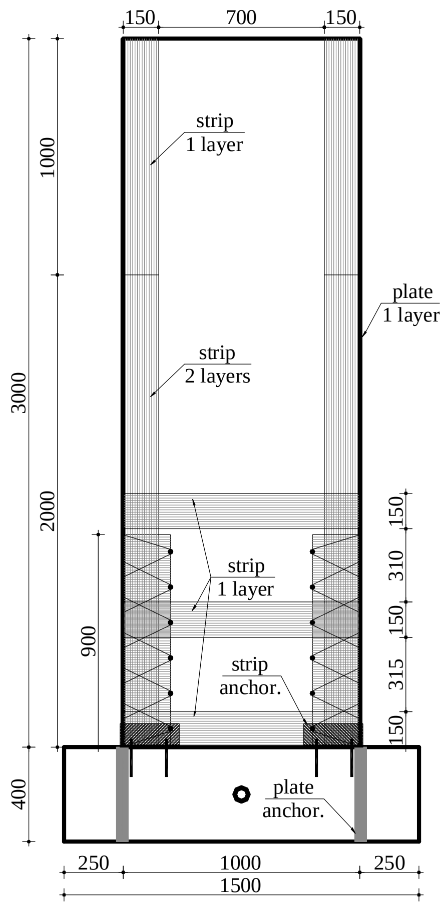
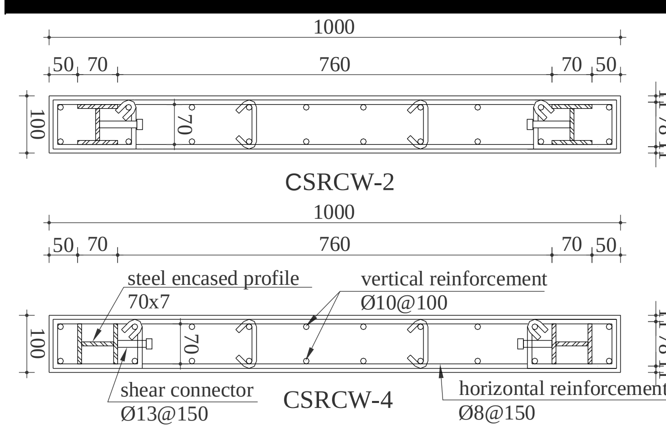
Experimental program

SPECIMENS

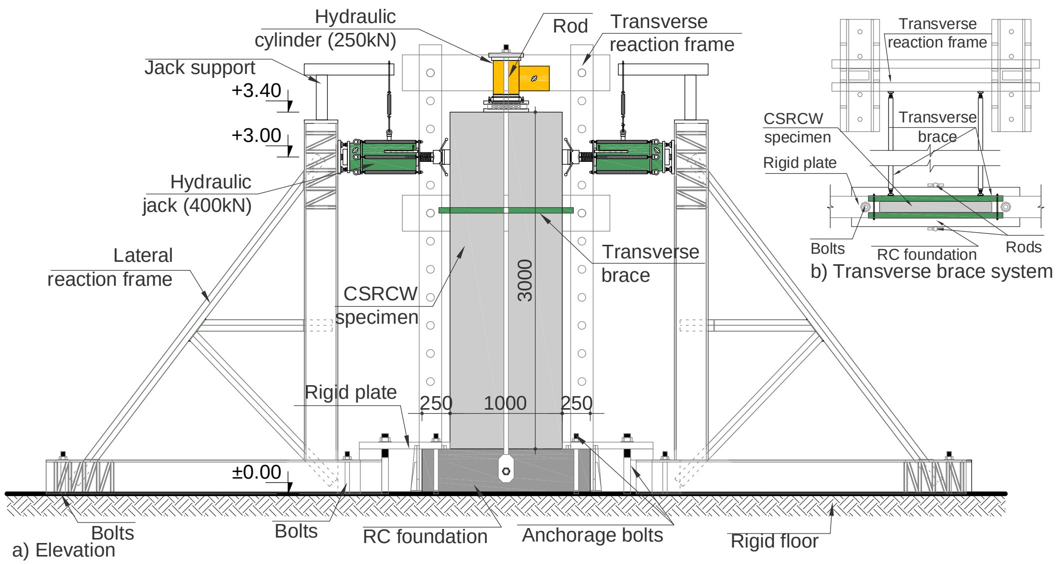
PLAIN CSRCW2 / CSRCW4	RETROFITTED POST-DAMAGE CSRCW2_R / CSRCW4_R

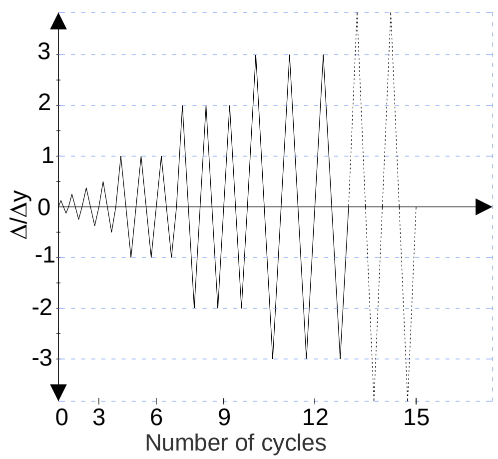


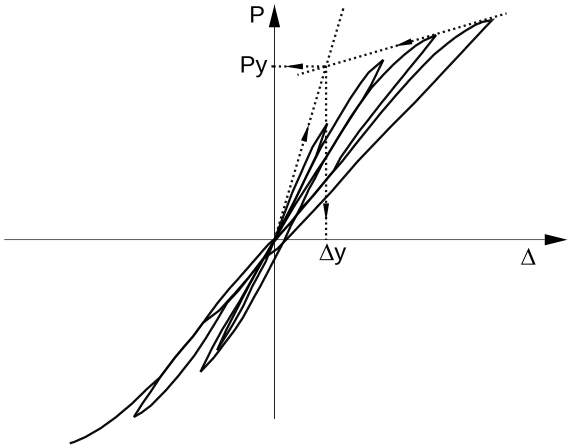




Experimental program TEST SET-UP AND LOADING STRATEGY







AXIAL

- N = 100 kN

LATERAL

- Drift increment: ECCS procedure
- Cycles: 3
- Failure: -15%

Experimental program

RETROFITTING PHASES

- **DESIGN OF THE RETROFITTING**

BASED ON THE PRINCIPLES (QUALITATIVE)

- FAILURE MECHANISM
- FIBRE ORIENTATION
- ANCHORAGES

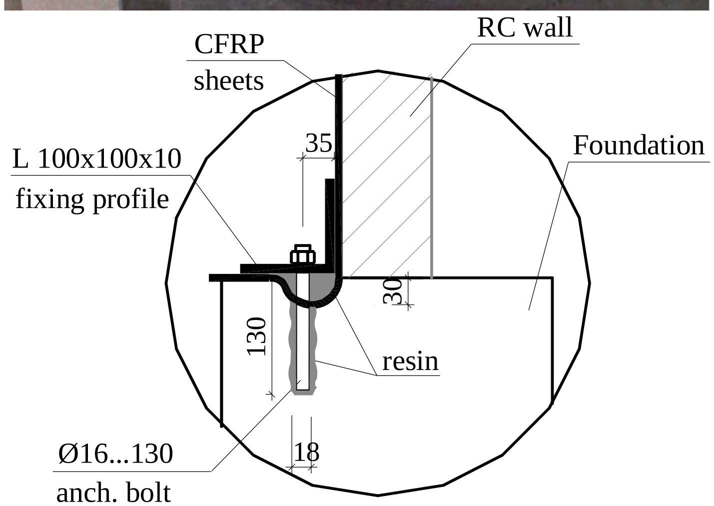
BASED ON THE DEMANDS (QUANTITATIVE)

- MATERIALS: Ef , tf , ff
- FIBRE SHEETS WIDTH: 150 mm AND 200 mm
- NUMBER OF LAYERS: 1 OR 2
- FIBRE PLATES WIDTH: 50 mm

- **RETROFITTING**

- REPAIRING OF THE CRUSHED CONCRETE
- PREPARING OF THE CONCRETE SURFACE

Experimental program



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RETROFITTING

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Experimental program

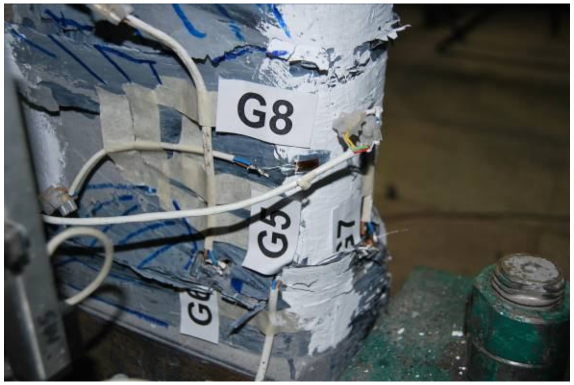
BEHAVIOR AND FAILURE MODES **CSRCW2_R**

1. FLEXURAL AND SHEAR CRACKS
2. FRP DEBONDING
3. FRP FRACTURING
4. REINFORCEMENT FRACTURE
5. CONCRETE CRUSHING

Experimental program

BEHAVIOR AND FAILURE MODES **CSRCW4_R**

1. FLEXURAL AND SHEAR CRACKS
2. FRP DEBONDING
3. FRP FRACTURING
4. REINFORCEMENT FRACTURE
5. CONCRETE CRUSHING



Analysis of the results Load – displacement responses

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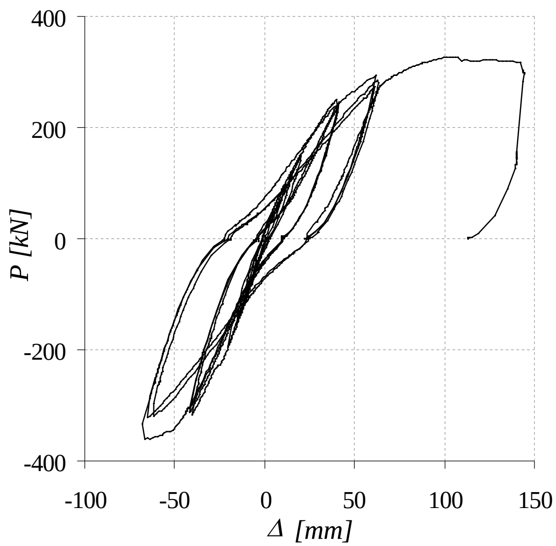

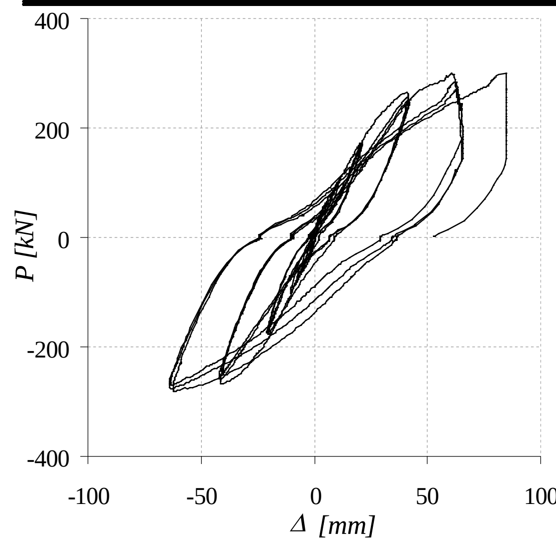

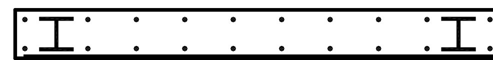

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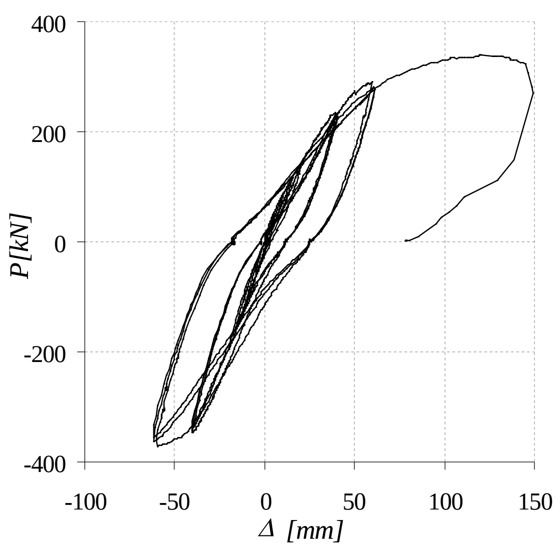

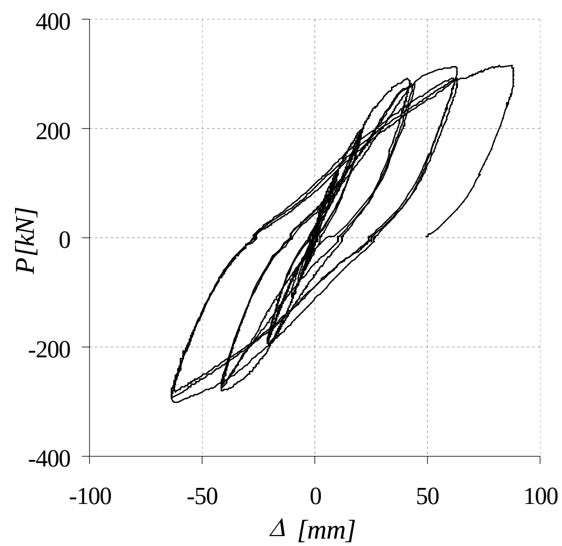

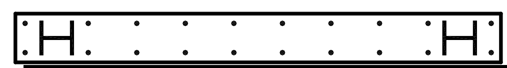

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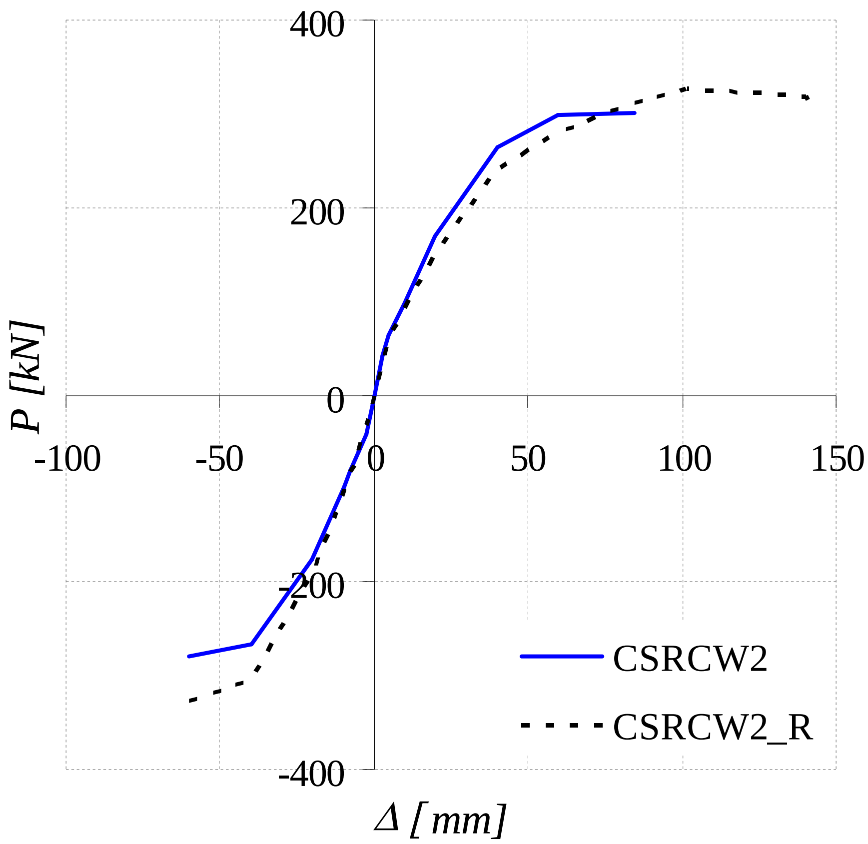
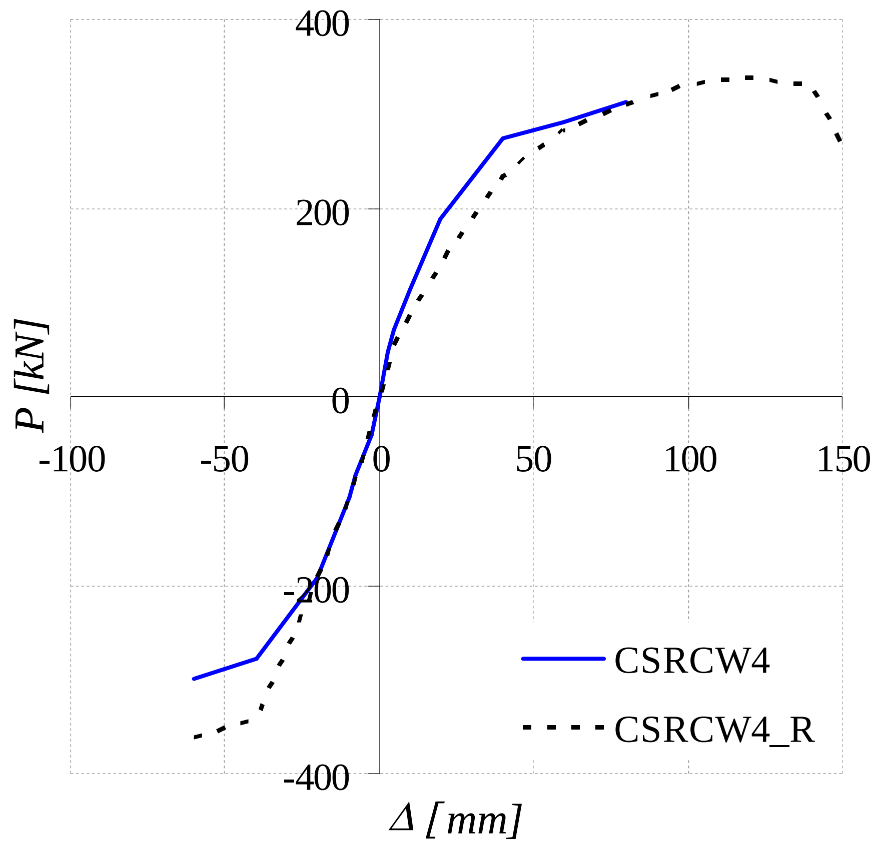
Analysis of the results

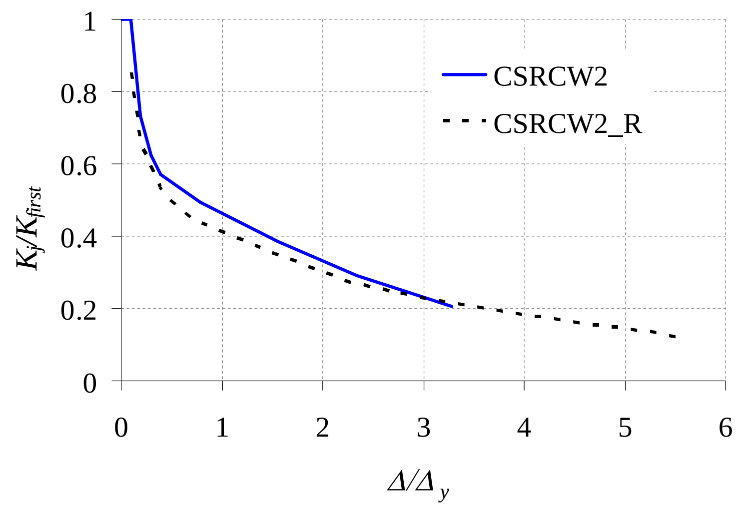
BEHAVIOR COMPARISON
LOAD – DISPLACEMENT ENVELOPE CURVES

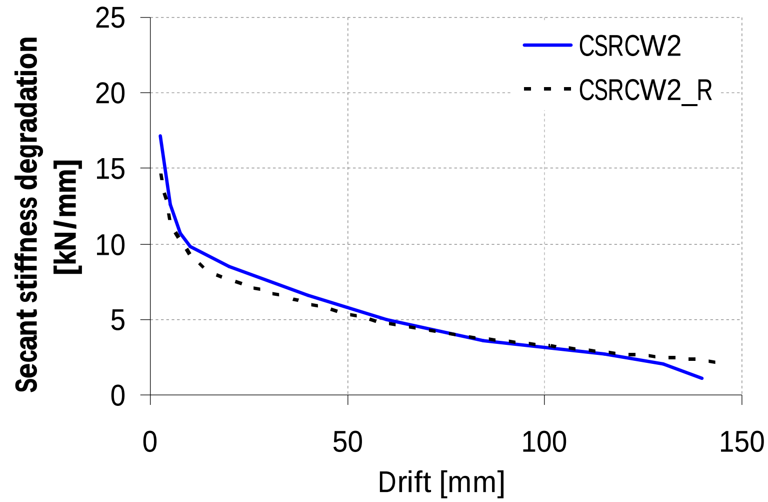


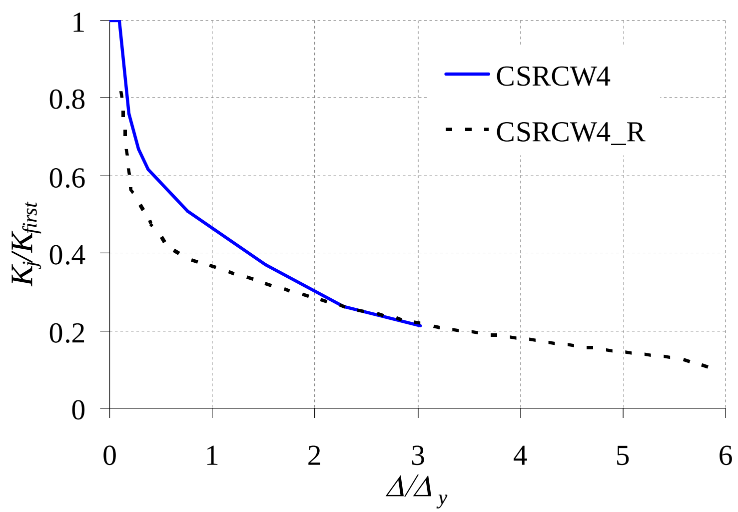


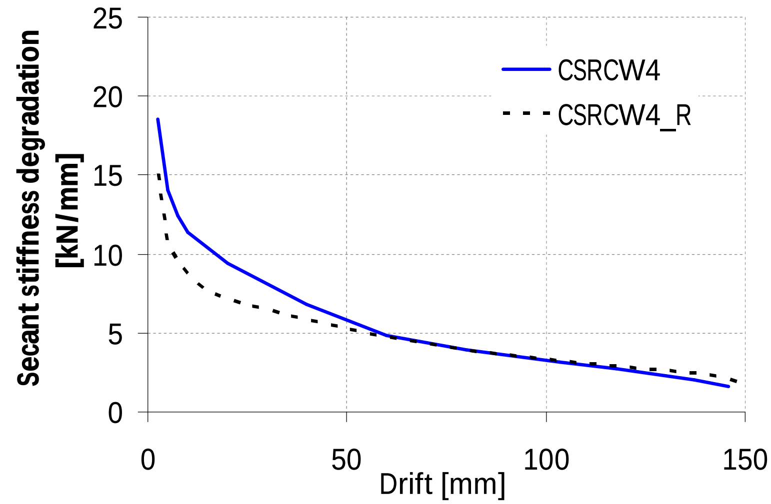
Analysis of the results

BEHAVIOR COMPARISON STIFFNESS DEGRADATION



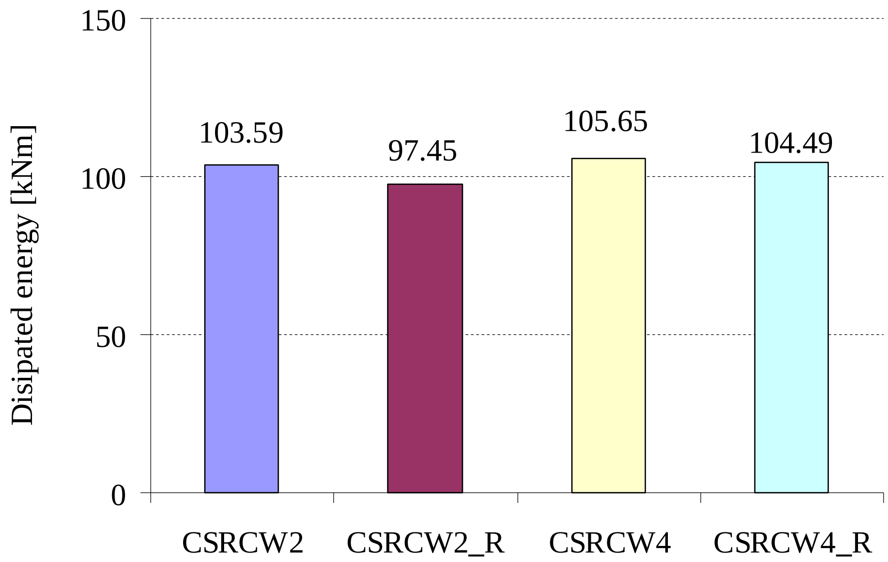


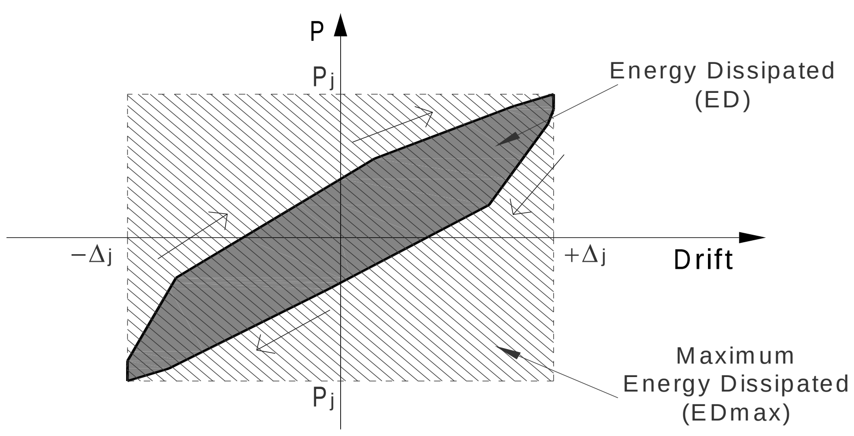




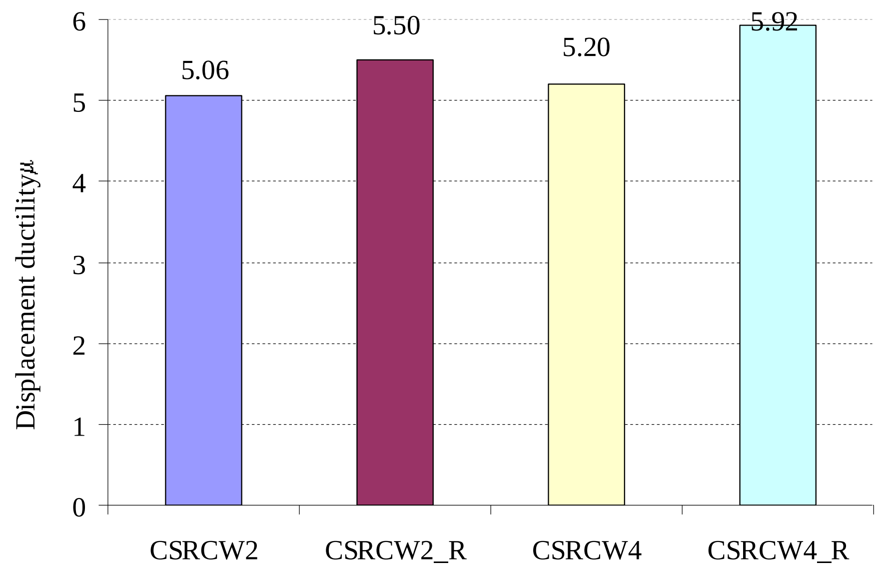
Analysis of the results

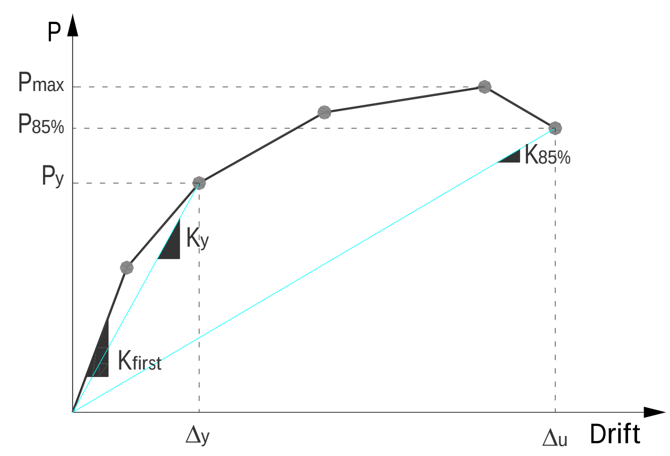
DISSIPATED ENERGY





DISPLACEMENT DUCTILITY





$$\mu = \frac{\Delta_u}{\Delta_y}$$

Conclusions

SEISMIC PERFORMANCE

- CFRP EBR RETROFITTING
- OVERALL STRENGTHENING WAS EFFECTIVE
- COMPONENT CONTRIBUTION TO THE PERFORMANCE WAS DIFFERENT

  - FLEXURAL STRIPS AND PLATES FAILED UNDER TENSION-COMPRESSION REVERSALS
  - SUBSTRATE DEGRADATION RESULTED IN DEBONDING OF THE SHEAR STRIPS AT INCLINED CRACK INTERSECTION
  - CONFINEMENT STRIPS REACHED THEIR ULTIMATE CAPACITY AND FAILED BY FIBER RUPTURE

Conclusions

1. EXPERIMENTAL RESEARCH PERFORMED ON COMPOSITE STEEL CONCRETE WALLS

→ TO RESTORE THE CAPACITY CAUSED BY DAMAGES

→ TO INVESTIGATE THE EFFECTIVENESS OF THE EXTERNALLY BONDED CFRP COMPOSITE MATERIALS AS RETROFITTING SOLUTION.

2. THE STRENGTHENING SOLUTIONS FOR CSRCW USING CFRP COMPOSITES ARE EFFICIENT IN TERMS OF RESTORING THE LOAD BEARING CAPACITY

3. THE INITIAL STIFFNESS OF THE RETROFITTED ELEMENTS WAS ABOUT 80% OF THE INITIAL STIFFNESS OF THE REFERENCE ELEMENTS AND DECREASED MORE RAPIDLY

4. THE ENERGY DISSIPATION CAPACITY UNTIL THE FRACTURE STAGE IS SMALLER FOR THE RETROFITTED ELEMENTS

5. THE ANCHORAGE PROVIDED FOR CFRP STRIPS AND PLATES WERE EFFICIENT

6. CERTAIN LIMITATIONS WERE IDENTIFIED ON THE USE OF THE EXTERNALLY BONDED CFRP SHEETS IN REVERSED CYCLIC APPLICATIONS.

RETROFIT OF REINFORCED CONCRETE SHEAR WALLS WITH CFRP COMPOSITES

- ***BY***

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- ***Janos GERGELY, Ph.D., Assoc. Prof., UNC CHARLOTTE, USA***
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OBJECTIVES

⇒ **EXPERIMENTAL STUDY THE EFFECTIVENESS OF CFRP COMPOSITES FOR THE SEISMIC RETROFIT OF R.C. SHEAR WALLS WITH STAGGERED OPENINGS**

⇒ **BEHAVIOUR OF THE ELEMENTS**

⇒ **ANCHORAGE DETAIL**

⇒ **STRENGTHENING SYSTEM**

⇒ **CHANGES IN STIFFNESS AND DUCTILITY**

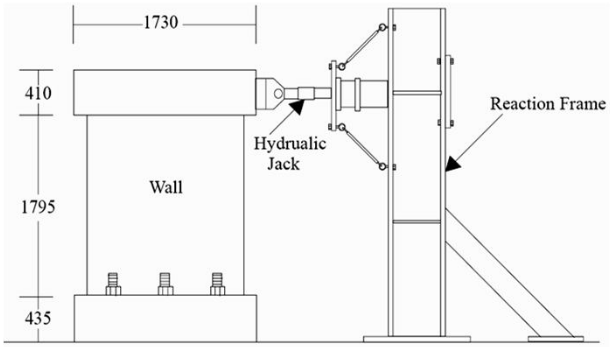
⇒ **COLLAPSE MECHANISM**

WHY R.C. WALL STRENGTHENING WITH CFRP ?

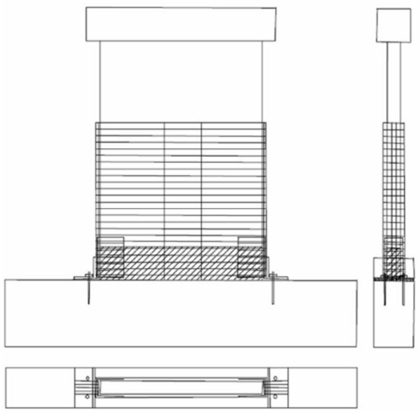
⇒ **NUMEROUS APPLICATIONS WORLDWIDE**

⇒ **FEW RESEARCH RESULTS**

⇒ **REDUCED KNOWLEDGE**



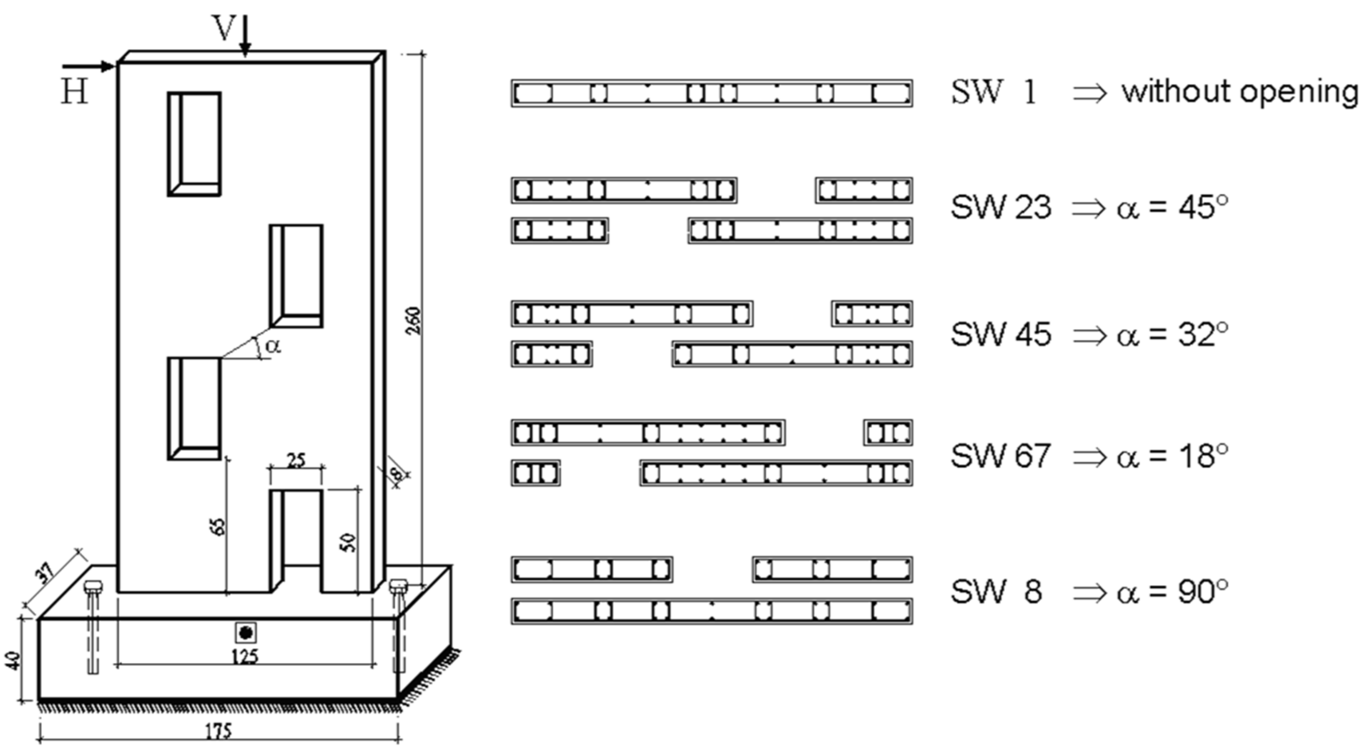
- LOMBARD et all. (2000) – 4 walls



- ANTONIADES et all. (2003) – 6 + 5 walls

EXPERIMENTAL PROGRAM

⇒ 5 SHEAR RC WALLS SPECIMENS, SCALE 1:4

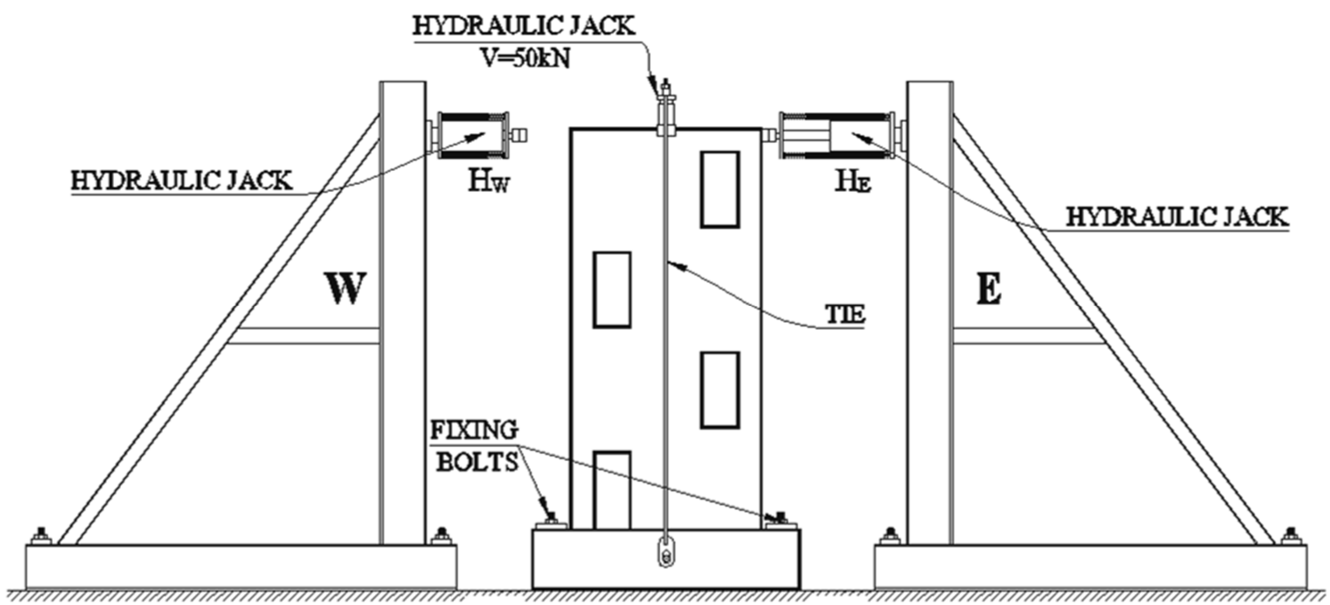


EXPERIMENTAL PROGRAM

1. TESTING THE BASELINE WALLS UP TO FAILURE

V = 50 KN → constant vertical load

H_E and H_W → monotonic increased horizontal load in a displacement-controlled mode



EXPERIMENTAL PROGRAM

1. TESTING THE BASELINE WALLS UP TO FAILURE

⇒ RESULTS

- SW1 (without openings) → TYPICAL BENDING BEHAVIOUR

- SW8 → PLASTIC HINGE → IN COUPLING BEAMS
→ AT THE BASE

- SW23, SW45, SW67 → CONCRETE CRUSHING AT THE BASE OF THE SMALL SIDEWALL
→ CANTILEVER-LIKE BEHAVIOUR OF THE BIGGER SIDEWALL

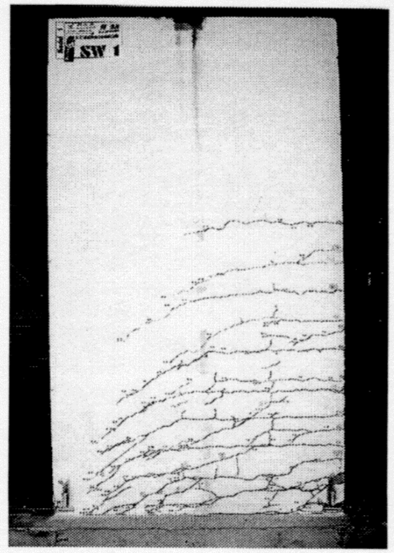
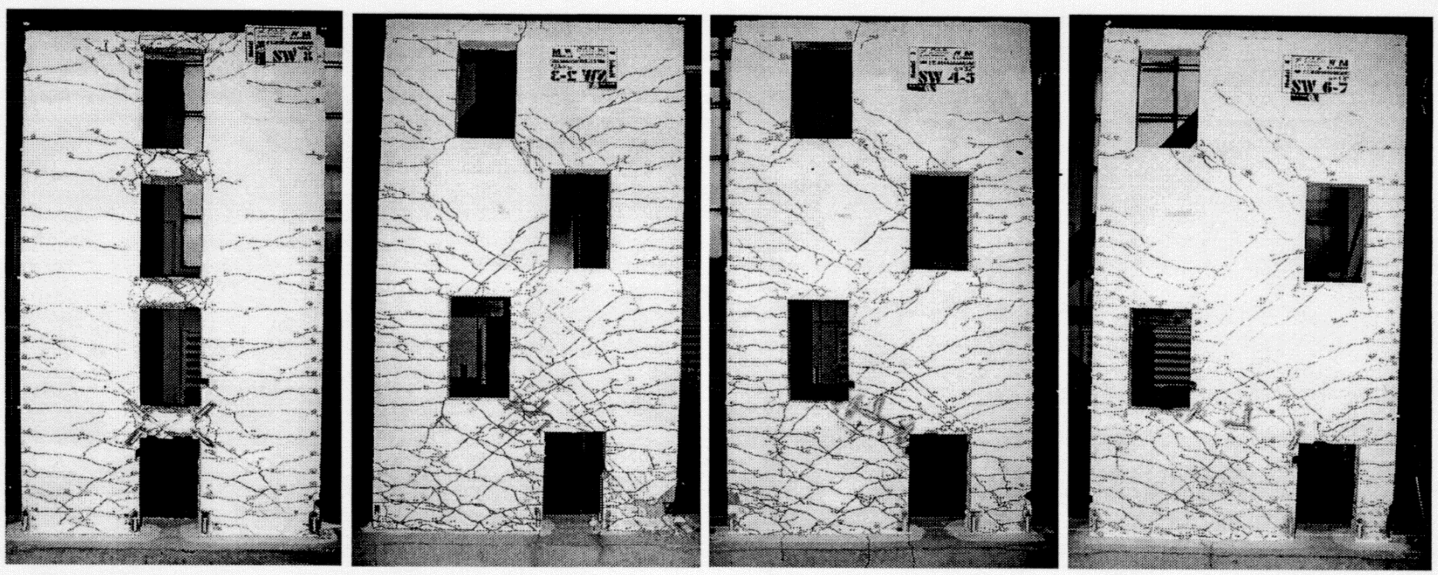


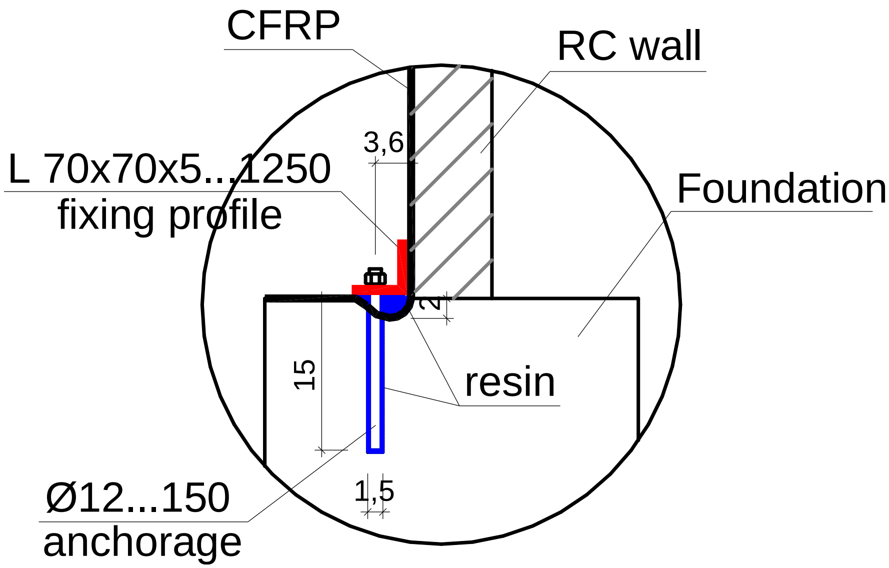
Fig.6 Cedarea modelului SW1



EXPERIMENTAL PROGRAM

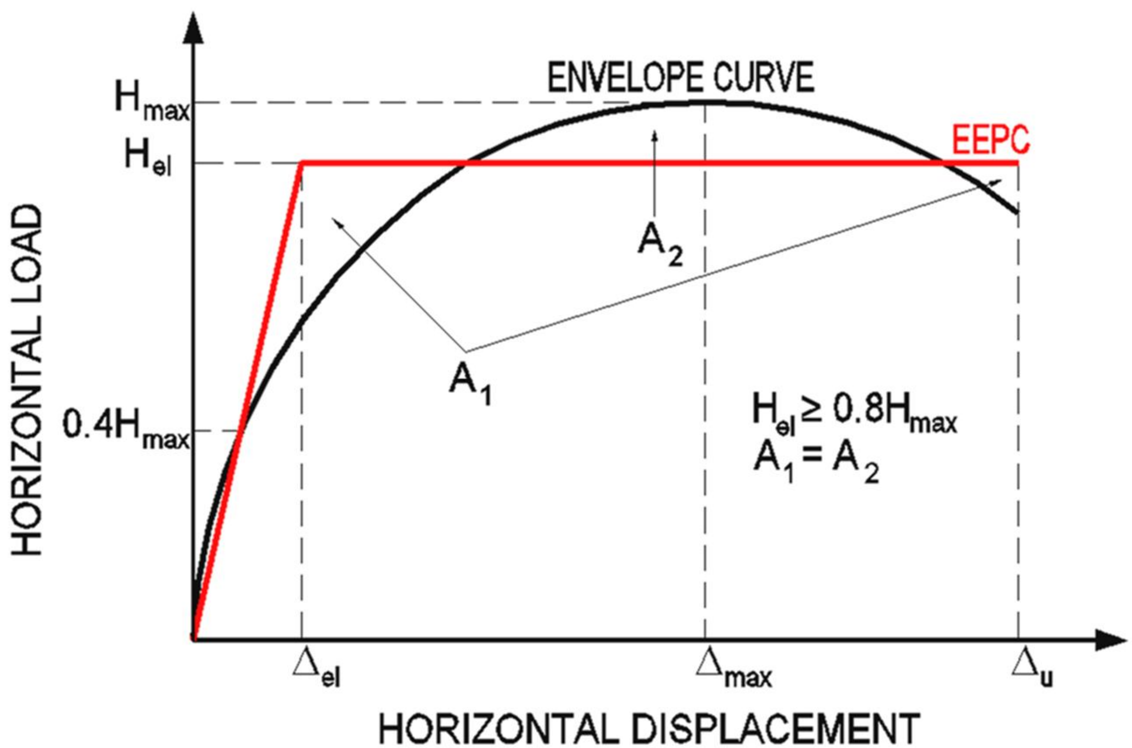
2. RETROFITTING OF THE WALLS → UNIDIRECTIONAL CFRP FABRIC ON ONE SIDE

- **REPLACE THE DAMAGED PARTS WITH EPOXY MORTAR**
- **FILLING CRACKS WITH AN EPOXY RESIN**
- **CLEANING THE SURFACES**
- **ANCHORAGE OF THE CFRP**



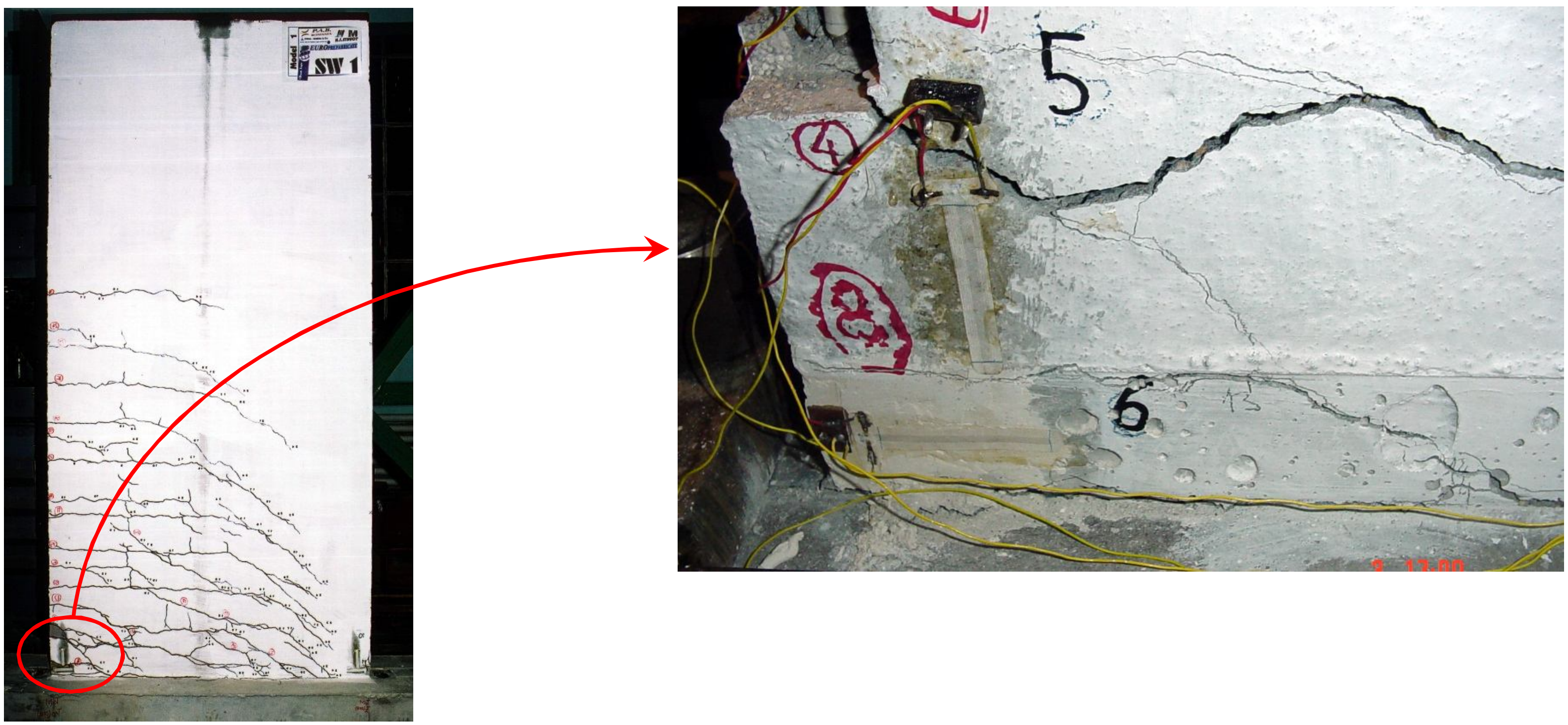
EXPERIMENTAL PROGRAM

3. PROCEDURE FOR DETERMINING THE EQUIVALENT ELASTO-PLASTIC CURVE (EEPC)→ STIFFNESS AND DUCTILITY



EXPERIMENTAL PROGRAM

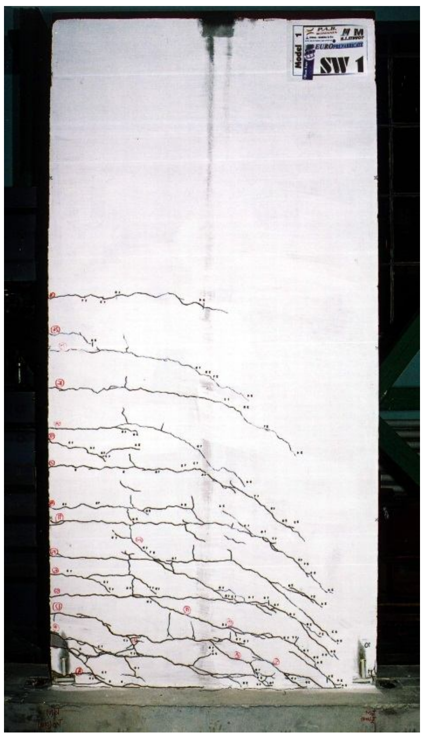
4. TEST RESULTS

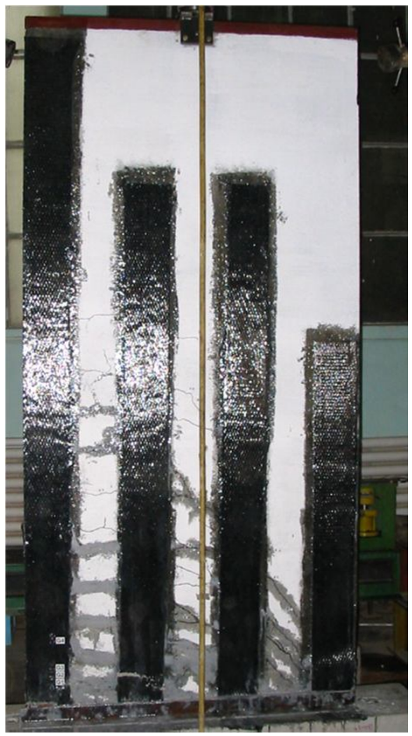


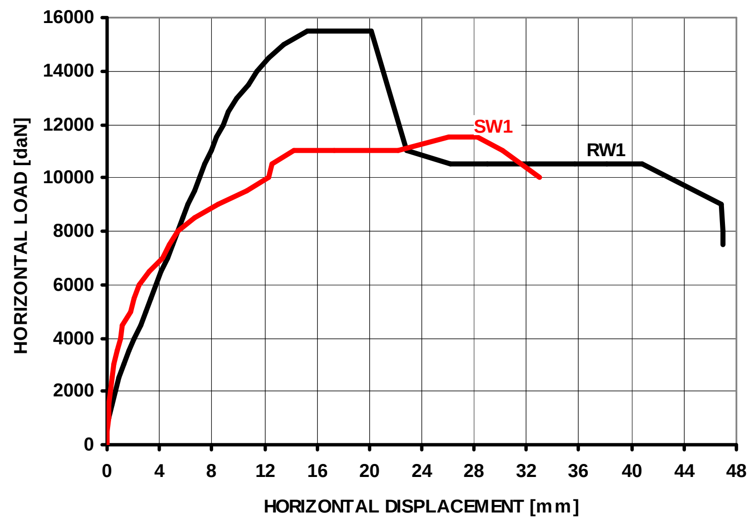
EXPERIMENTAL PROGRAM

4. TEST RESULTS

SW1 **RW1**







μ =11.2 **μ =4.8**

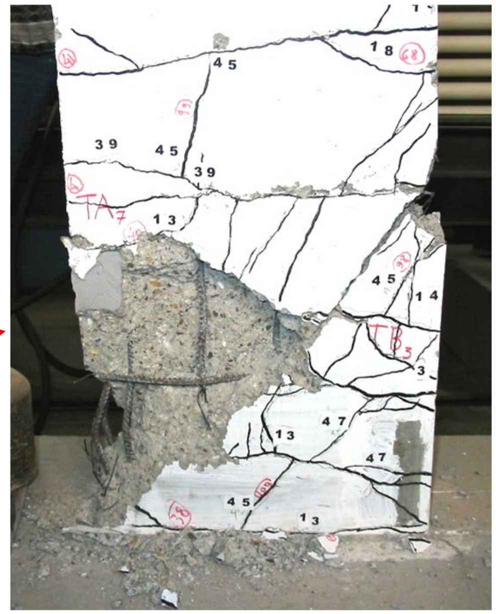
Difference in capacity : +35%
Difference in displacement: +42%
Strain in composite : 0.54%

RW1 : CFRP TENSION FAILURE → CONCRETE CRASHING AND CFRP DEBONDING IN COMPRESSION

EXPERIMENTAL PROGRAM

4. TEST RESULTS

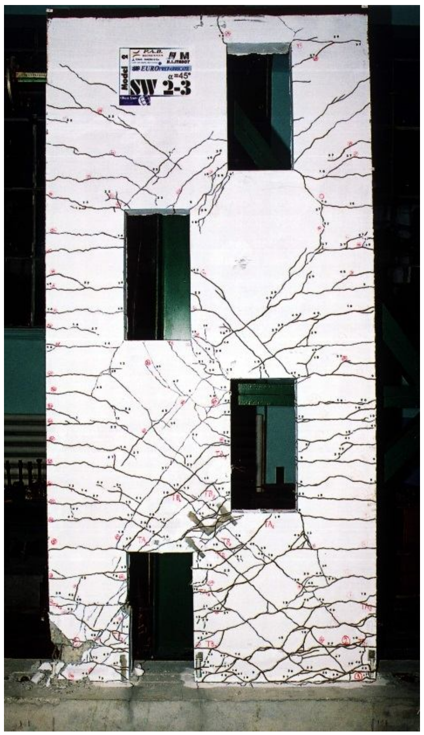
SW23

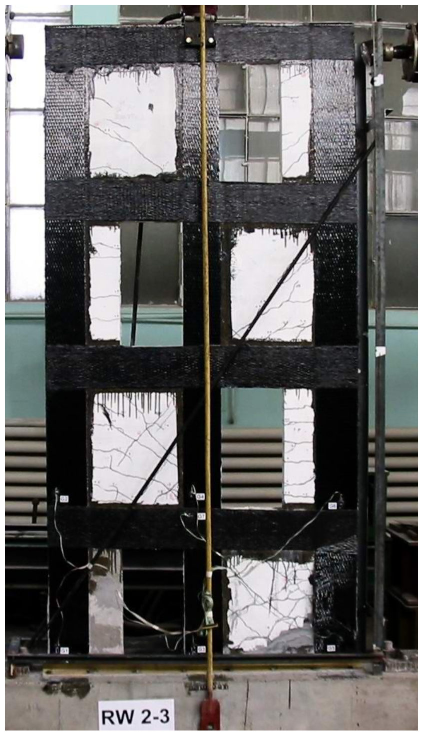


EXPERIMENTAL PROGRAM

4. TEST RESULTS

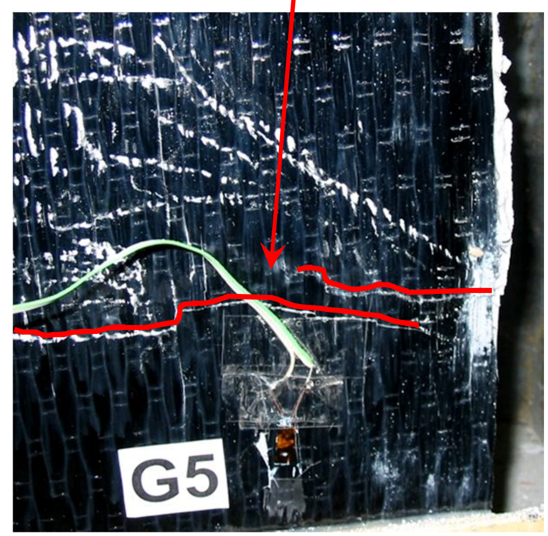
SW23 **RW23**





DEBONDING

CRACKING IN COMPRESSION

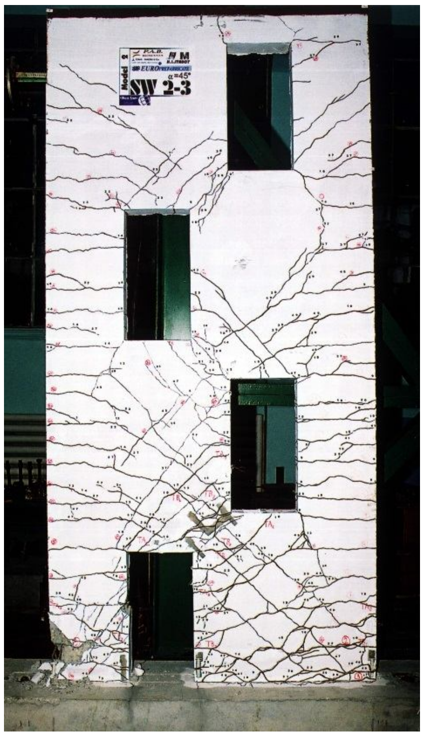


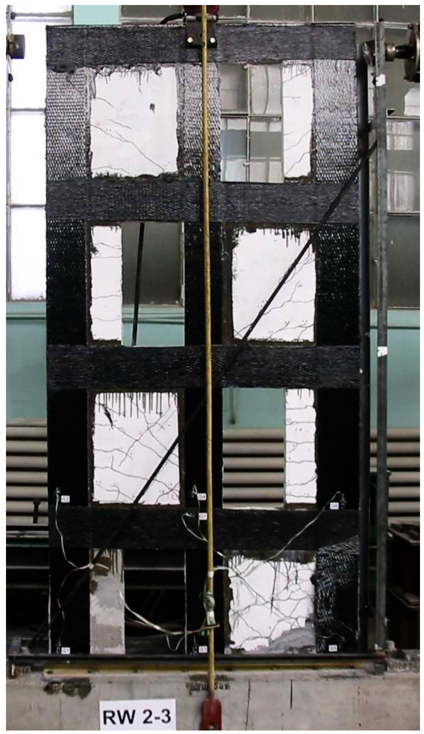
RW23 :
- → GRADUALLY OPENING OF THE CRACKS
- → CFRP DEBONDING & CRACKING IN COMPRESSION
- → CFRP TENSION FAILURE

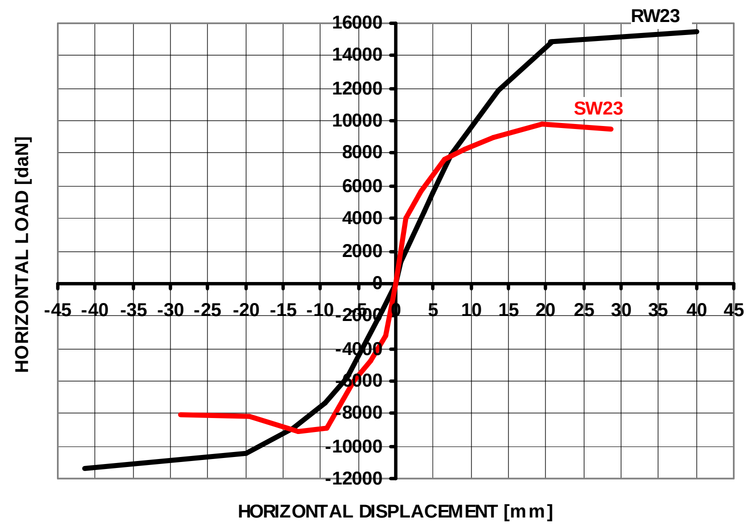
EXPERIMENTAL PROGRAM

4. TEST RESULTS

SW23 **RW23**







μ_W =10.2 / Δ_E = 6.6 **μ_W = 3.0 / μ_E = 3.6**

	W // E
Difference in capacity :	**+58% // +22%**
Difference in displacement :	**+42% // +46%**
Strain in composite :	**0.63%**

EXPERIMENTAL PROGRAM

4. TEST RESULTS

SW45

EXPERIMENTAL PROGRAM

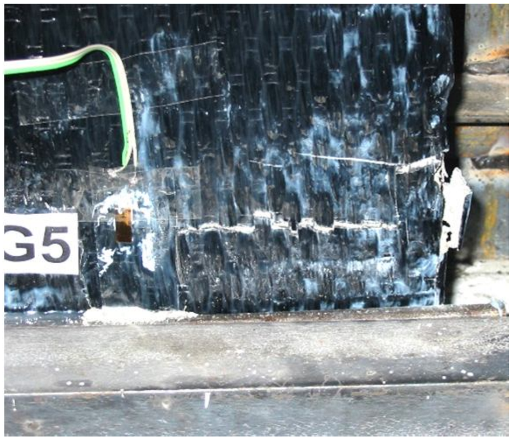
4. TEST RESULTS

SW45

RW45

DEBONDING

TENSION FAILURE



RW45 : → GRADUALLY OPENING OF THE CRACKS
→ CFRP DEBONDING & CRACKING IN COMPRESSION
→ CFRP TENSION FAILURE & CONCRETE CRUSHING

EXPERIMENTAL PROGRAM

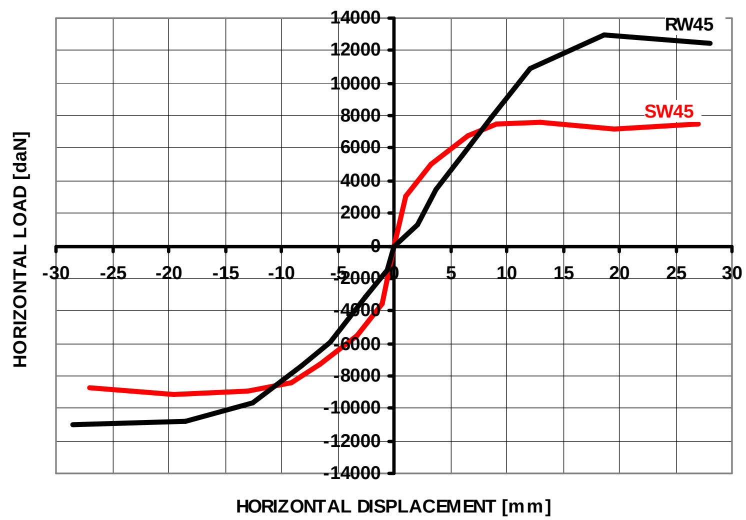
4. TEST RESULTS

SW45

$\mu_W = 11.5$ / $\mu_E = 11.3$

RW45

$\mu_W = 2.1$ / $\mu_E = 3.2$

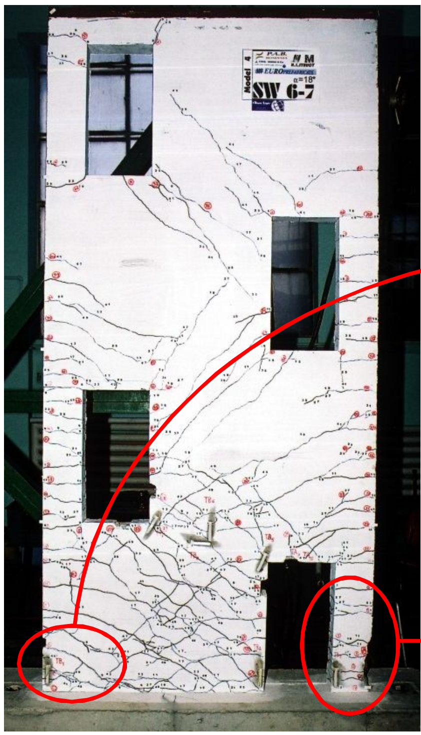


	W // E
Difference in capacity :	**+71% // +19%**
Difference in displacement :	**+1% // +1%**
Strain in composite :	**0.79%**

EXPERIMENTAL PROGRAM

4. TEST RESULTS

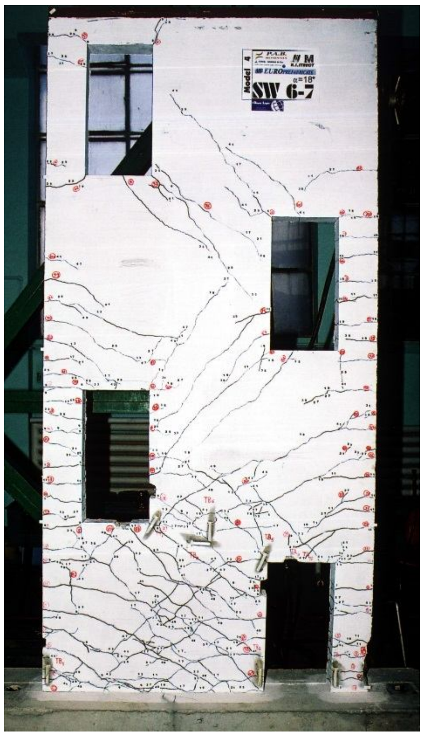
SW67



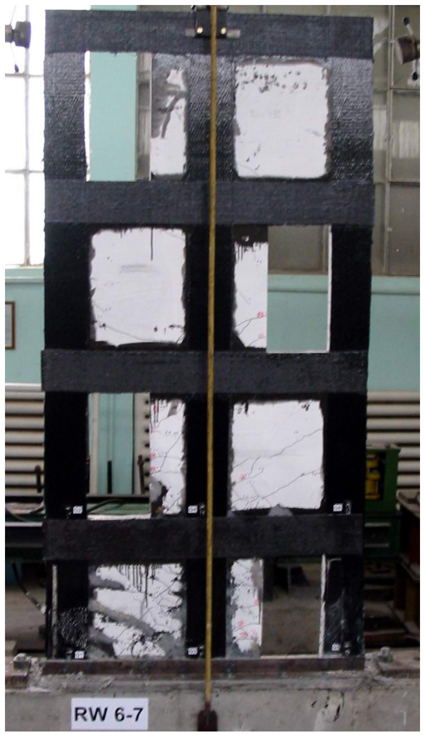
EXPERIMENTAL PROGRAM

4. TEST RESULTS

SW67

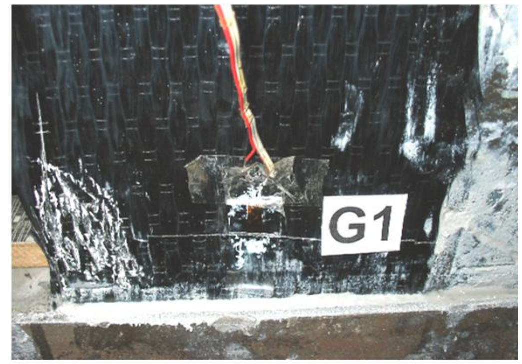


RW67



DEBONDING

CRACKING IN COMPRESSION



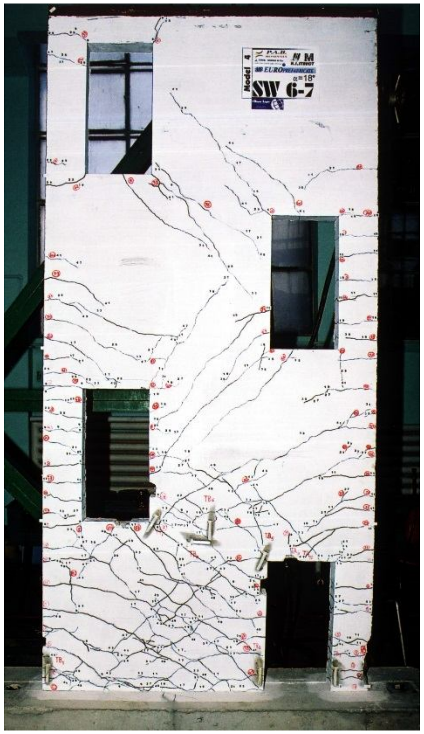
TENSION FAILURE

RW67 : → GRADUALLY OPENING OF THE CRACKS
→ CFRP DEBONDING & CRACKING IN COMPRESSION
→ CFRP TENSION FAILURE & CONCRETE CRUSHING

EXPERIMENTAL PROGRAM

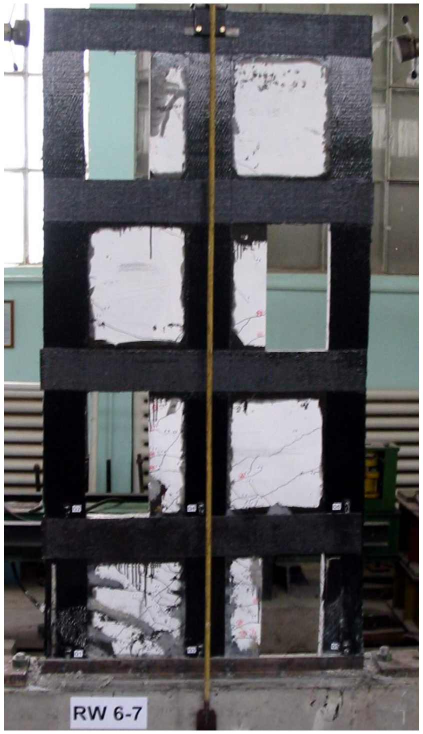
4. TEST RESULTS

SW67

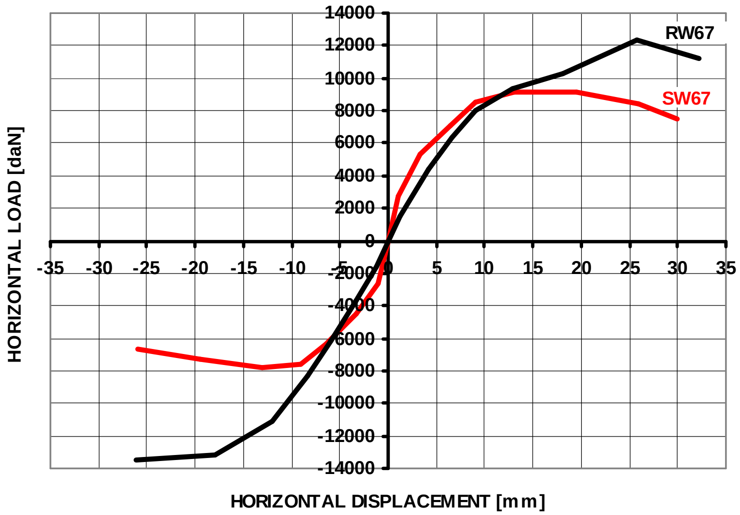


$\mu_W = 7.1$ / $\mu_E = 6.3$

RW67



$\mu_W = 2.2$ / $\mu_E = 3.1$

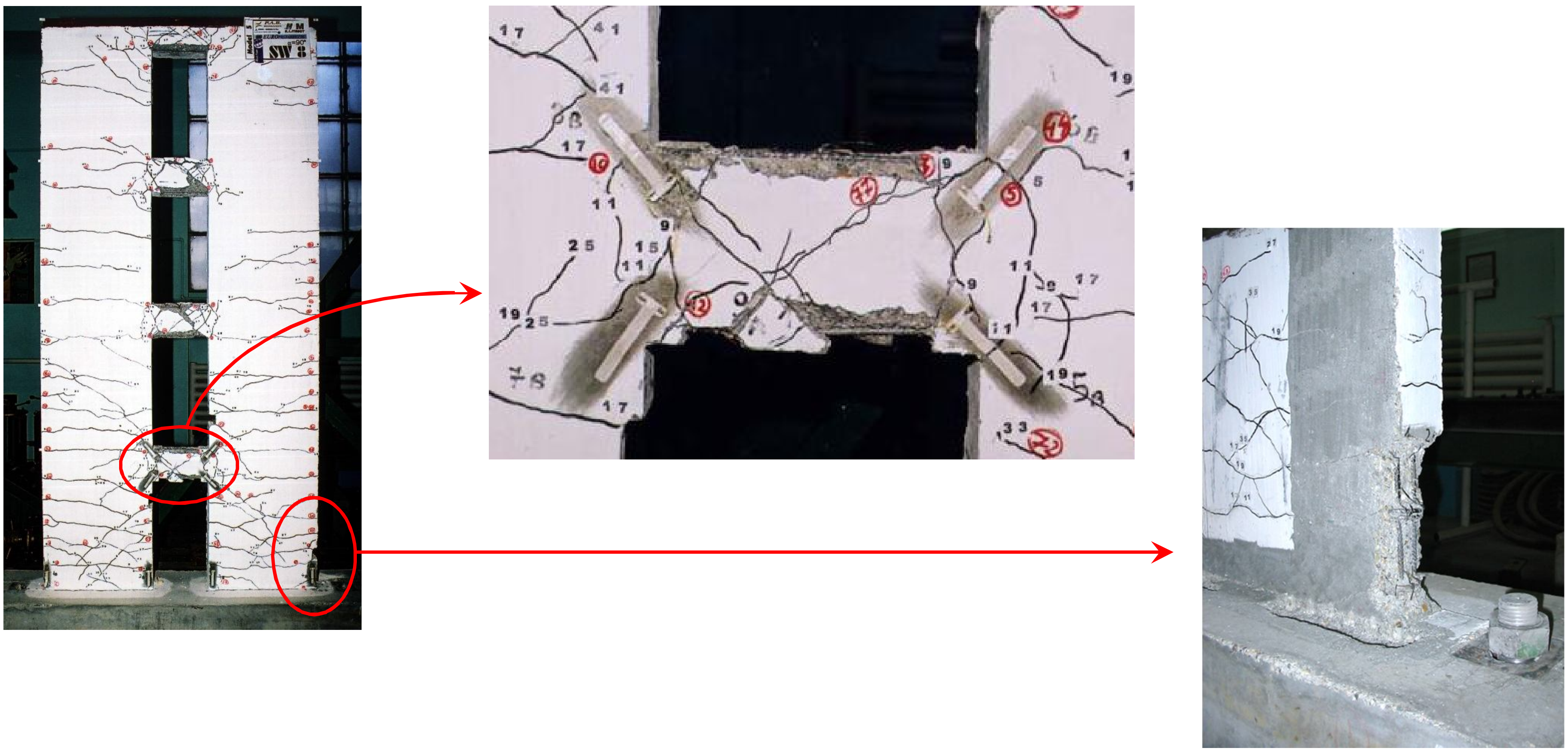


	W // E
Difference in capacity :	**+48% // +57%**
Difference in displacement :	**+6% // +0%**
Strain in composite :	**0.58%**

EXPERIMENTAL PROGRAM

4. TEST RESULTS

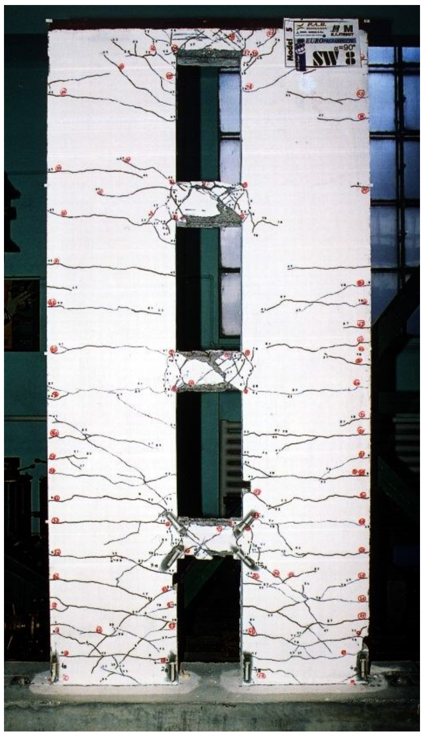
SW8



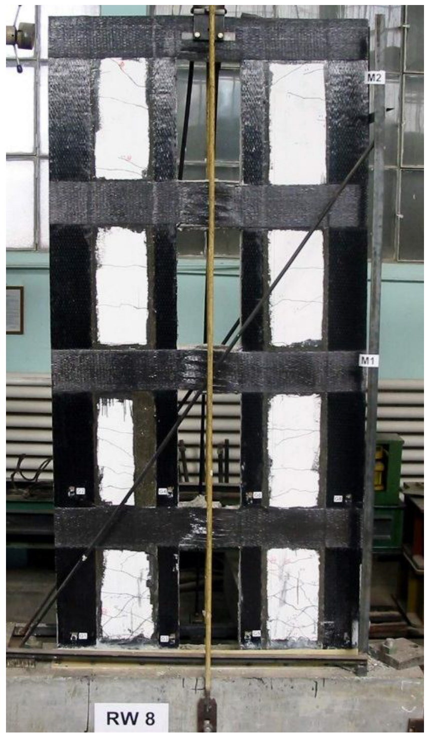
EXPERIMENTAL PROGRAM

4. TEST RESULTS

SW8



RW8



DEBONDING

TENSION IN COUPLING BEAM

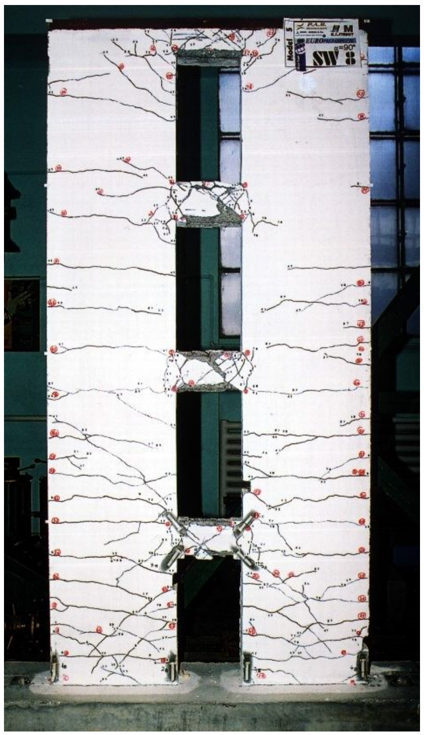
TENSION FAILURE

RW8 :
- → GRADUALLY OPENING OF THE CRACKS
- → CFRP DEBONDING IN COMPRESSION
- → CFRP TENSION FAILURE
- → CRACKING & FAILURE OF COUPLING BEAMS

EXPERIMENTAL PROGRAM

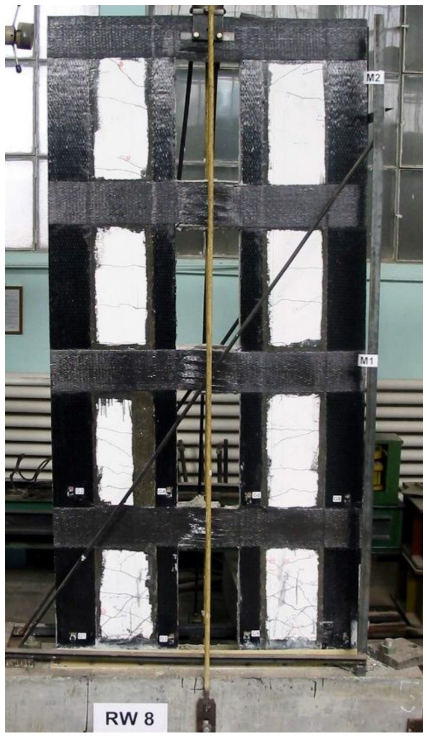
4. TEST RESULTS

SW8

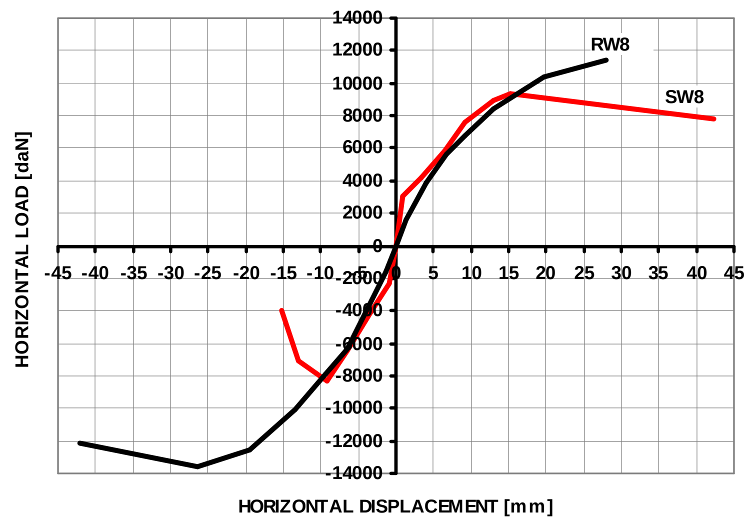


μ_W =7.9 / μ_E =3.2

RW8



μ_W =2.6 / μ_E =3.4



	W // E
Difference in capacity :	**+21% // +63%**
Difference in displacement :	**-33 % // +180%**
Strain in composite :	**0.83%**

CONCLUSIONS

✶ **THE LOAD BEARING CAPACITY OF THE RETROFITTED WALLS INCREASED CONSIDERABLY OVER THE BASELINE VALUE**
⇒ **practically, the load bearing capacity of the pre-tested walls was negligible**

✶ **THE FAILURE OF THE RETROFITTED WALL WAS CAUSED BY:**

1. **gradually opening of the existing cracks**
2. **debonding of the FRP in compression**
3. **cracking of the FRP in compression followed by tension failure**
4. **concrete crashing**

✶ **THE RESULTS WERE HIGHLY DEPENDENT BY THE INITIAL STATE OF THE RETROFITTED ELEMENT (width and number of cracks, yielded steel reinforcement, rehabilitation method and material), RESPECTIVELY BY THE EVALUATION METHOD.**

CONCLUSIONS

✶ **BASED ON THE USED METHOD :**

- the elastic limit of the walls increased, in average, with 47%
- the failure load of the walls increased, in average, with 45%
- the stiffness of the elements decreased, in average, with 53%
- the ductility of the elements decreased, in average, with 60%

✶ **THE ANCHORAGE SYSTEM BEHAVED EXCELLENTLY, WITHOUT DEGRADATIONS.**

⇒ **RC WALLS SUBJECTED TO SEISMIC FORCES HAD A DUCTILE FAILURE. RETROFITTING SUCH STRUCTURAL ELEMENTS WITH COMPOSITES MAINTAIN THE CHARACTERISTIC DUCTILE BEHAVIOUR, BUT AT THE MAXIMUM LOAD FAILS BRITTLE.**

APPLICATION OF THE RESULTS

STRUCTURAL RETROFITTING OF PRECAST RC SLABS USIG FRP COMPOSITES

Valeriu STOIAN — *PhD, Professor*

István DEMETER — *PhD Student*

Sorin-Codruţ FLORUŢ — *PhD Student*

General outline of presentation

Aspects regarding structural retrofitting by means of FRP materials of:

I. Reinforced Concrete Floor Slab Panels

II. Precast Reinforced Concrete Wall Panels

(Reinforced Concrete shear walls)

- Experimentally investigate the behavior of Reinforced Concrete (RC) slabs subjected to flexure.
- Assess the overall alteration in behavior caused by inserting cut-out openings of various geometries.
- Investigate the effectiveness of retrofitting by externally bonded Carbon Fiber Reinforced Polymers (CFRP).
- Restoring, as much as possible, the strength and stiffness of elements with cut-outs up to the level of the full slabs.

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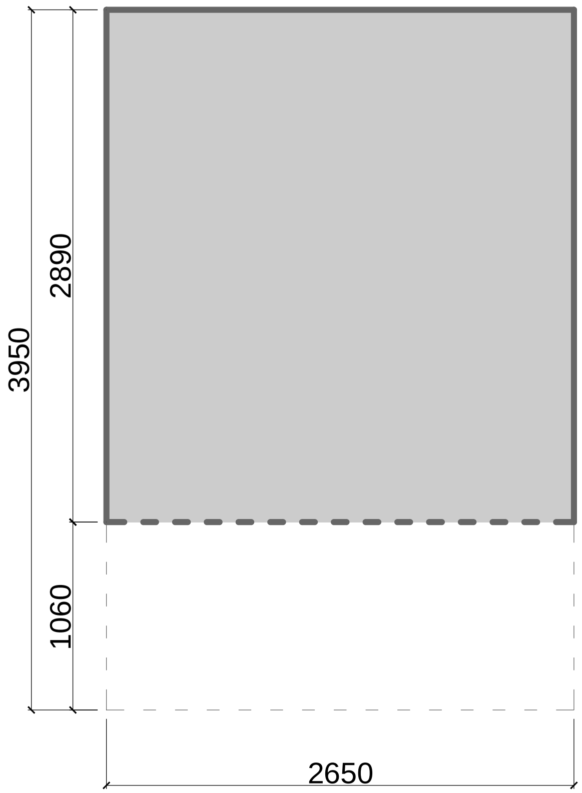
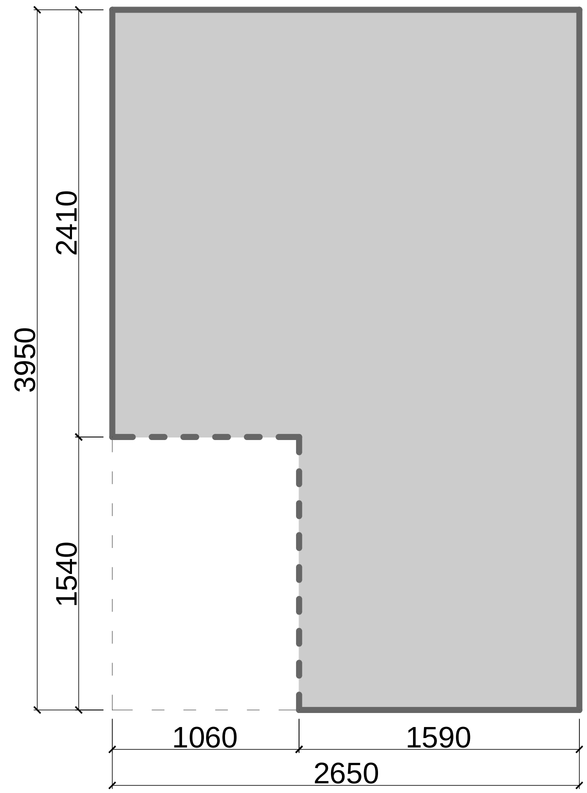
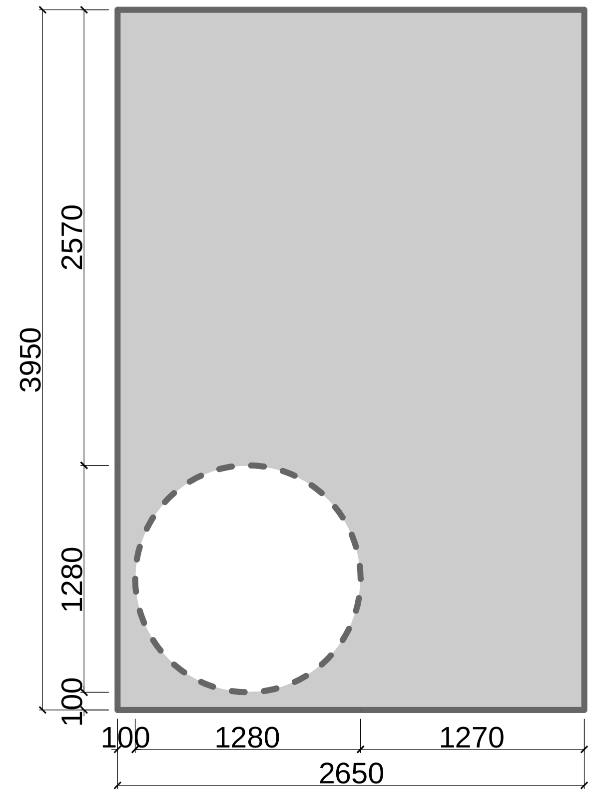


RCS-RLC-01



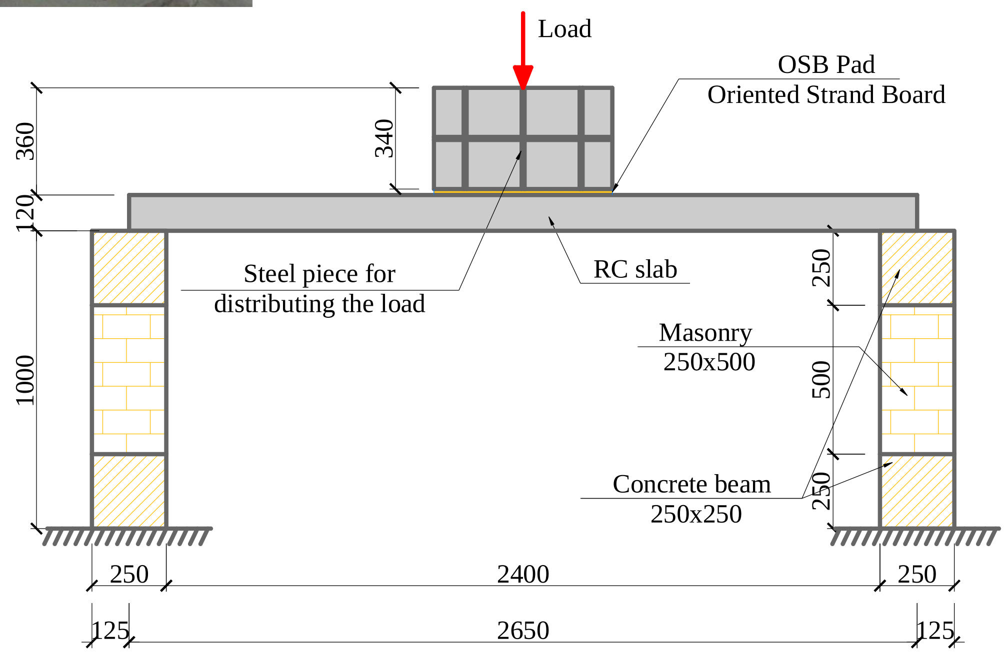
RCS-RSC-01



RCS-CC-01

- elements are simply supported on all of the four edges
- loaded gravitationally on a central patch of 600x1200 mm
- hydraulic jack with capacity of 500 kN and stroke of 160 mm
- before being retrofitted, all of the elements are to be tested up to a certain stage (that assumes the need of retrofitting). Afterwards, a mixed strengthening solution that involves the use of both near surface mounted reinforcement-fiber reinforced polymer (NSMR-FRP) and externally bonded-fiber reinforced polymers (EB-FRP) techniques is applied. After allowing the materials to cure, the elements will be retested up to failure.

PART A – STRUCTURAL REHABILITATION USING CFRP



CFRP retrofitting system

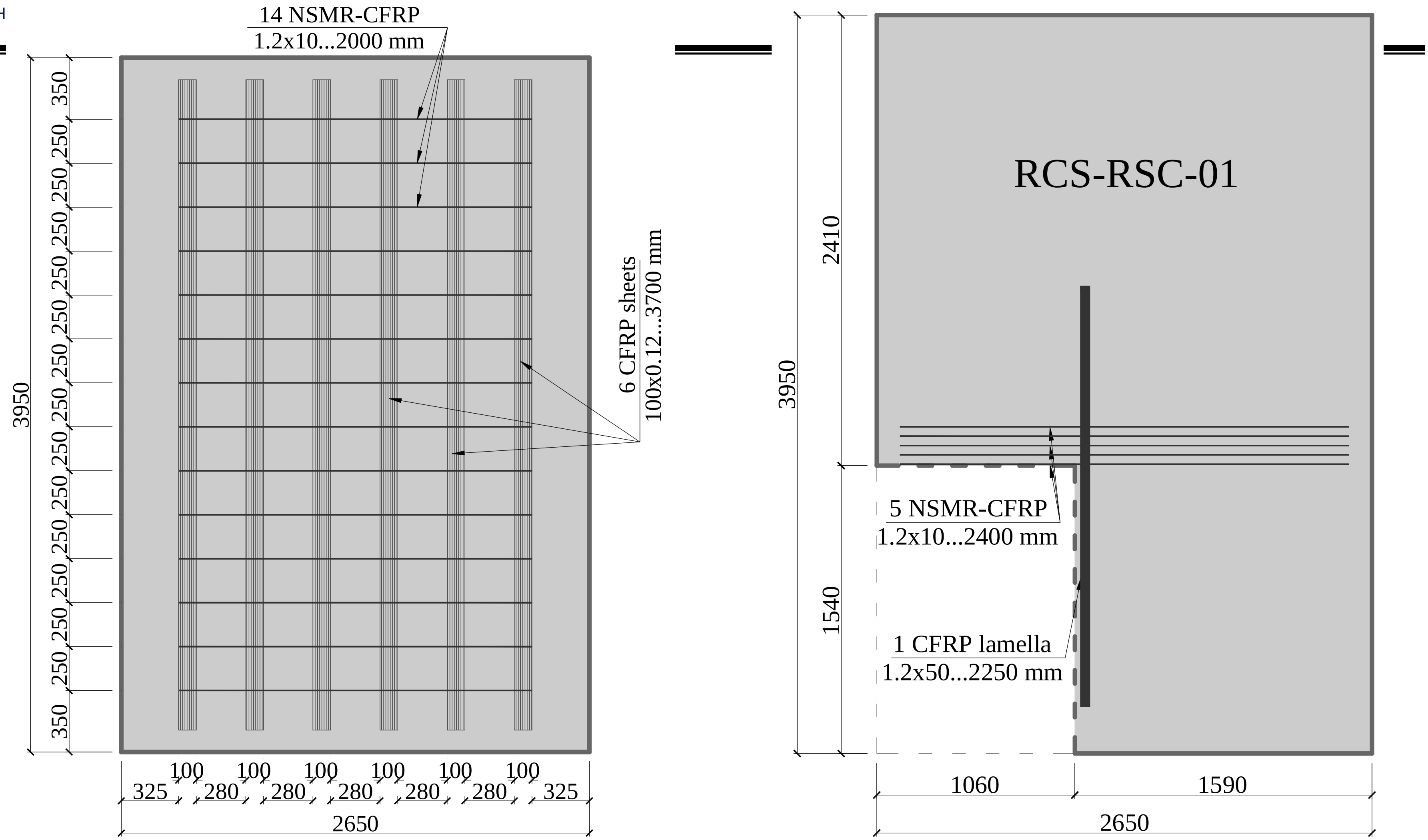
- CFRP components applied on the tensioned side
- combined near surface mounted reinforcement (NSMR-FRP) and externally bonded (EB-FRP) techniques.
- simplified design of retrofitting components based on two assumptions:

$$F_s = F_f \Rightarrow A_f = \frac{f_{yd}}{E_f \cdot \varepsilon_f} A_s$$

- for full slab - the required amount of CFRP is determined by equalizing the tension force that would have been undertook by the steel reinforcement (that is now yielded) and the tension force that will be undertook by the CFRP.

- for slab with cut-outs - the required amount of CFRP is determined by equalizing the tension force that would have been undertook by the steel reinforcement eliminated by creating the cut-out, and the tension force that will be undertook by the FRP.

PART A – STRUCTURAL REHABILITATION USING CFRP



Retrofitting solution for the full slab and the for the one with rectangular small cut-out opening

PART A – STRUCTURAL REHABILITATION USING CFRP

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PART A – STRUCTURAL REHABILITATION USING CFRP

- 14 NSMR-FRP parallel to the short edge (1.2x10…2000 mm)
- 6 EB-FRP parallel to the long edge (0.12x100…3700 mm)

- tests completed so far - full slab prior to retrofitting (RCS-FS-UU-01) and after retrofitting (RCS-FS-DS-01)
- RCS-FS-UU-01
  - max load level: 113 kN (past this value, the strain in numerous reinforcement bars has reached yielding point and the vertical mid-span displacement has past the maximum allowable deflection (L/250=2400/250=9.60 mm).
  - max strain in reinforcement was 7.52‰
  - max vertical mid-span displacement: 10.28 mm
  - four cracks appeared on the direction of the yield lines. The cracks originated at the corners of the load patch, being inclined at angles of 37°, 45°, 52°, and 55°.

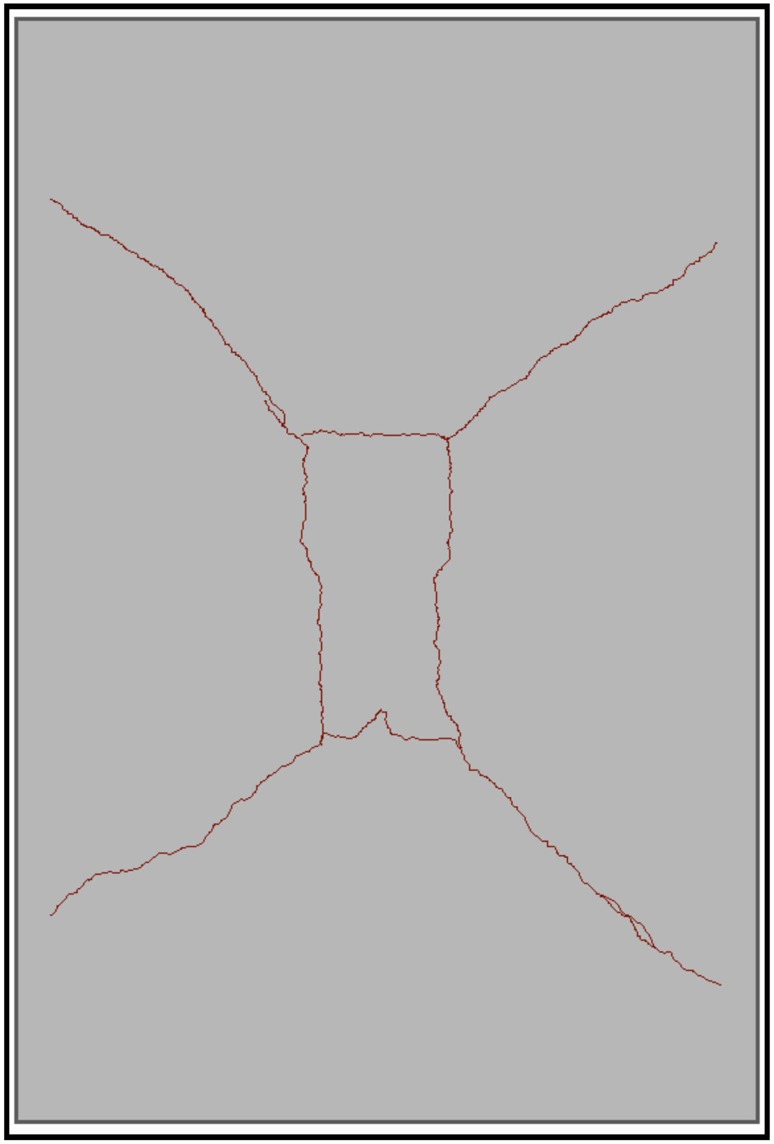
- RCS-FS-DS-01

  - max load level: 180 kN (corresponding to a vertical mid-span deflection of 50 mm

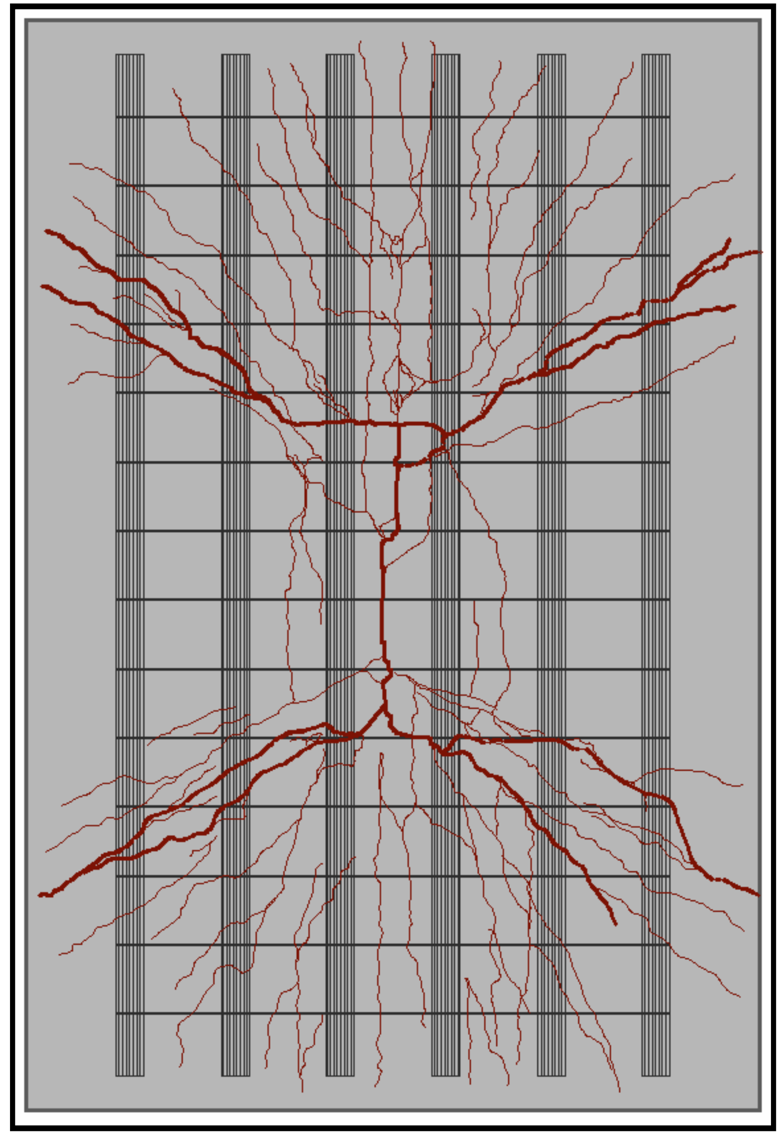
  - rupture in EB-FRP sheets and the NSMR-FRPs that were intersected by the principal cracks

  - max vertical mid-span displacement: 110 mm

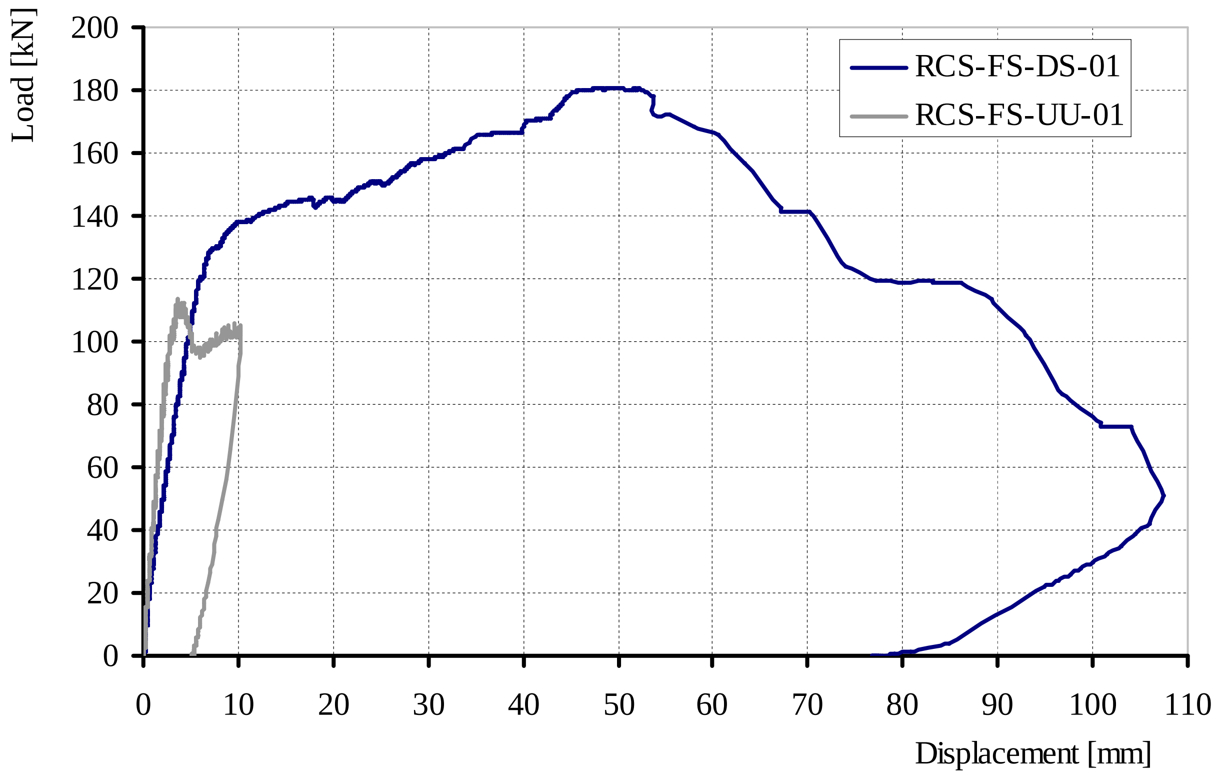
  - four cracks appeared on the direction of the yield lines. The cracks originated at the corners of the load patch, being inclined at angles of 37°, 45°, 52°, and 55°.



RCS-FS-UU-01



RCS-FS-DS-01


Load [kN]
200
180
160
140
120
100
80
60
40
20
0
0
10
20
30
40
50
60
70
80
90
100
110
Displacement [mm]
RCS-FS-DS-01
RCS-FS-UU-01

- The slab was able to deflect almost 110 mm before failure, thus giving an ample visual warning before collapse.

- The EB-FRP sheets and the NSMR-FRPs that were intersected by the principal cracks (the ones highlighted with thicker lines), have failed due to CFRP rupture, no type of premature failure (e.g. slipping, debonding, delamination, etc) being observed.

- The principal cracks in the case of the strengthened element are not necessarily on the same direction as the cracks that were opened after the test on the unretrofitted element.

- For the retrofitted element the high number of cracks, that appeared at various directions and are distributed on large areas of the slab, is in contrast with the unretrofitted elements, for which only the four cracks on the direction of the yield lines have opened.

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RETROFIT OF CUT-OUT WEAKENED PRECAST RC PANELS BY EXTERNALLY BONDED CFRP COMPOSITES

Valeriu STOIAN *Prof.,PhD.,Civ.Eng.*

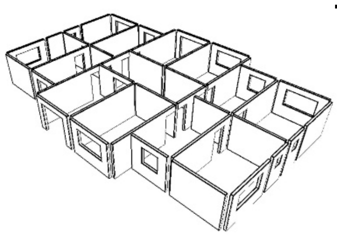
Daniel DAN *Assoc.prof.,PhD.,Civ.En g.*

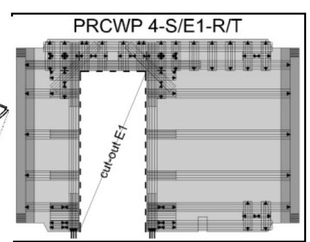
Tamás NAGY *Lect.,Ph.D.,Civ.Eng.*

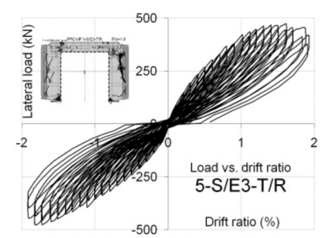
István DEMETER *Asist.Ph.D.,Civ.Eng.*

Codrut FLORUT *Asist.Ph.D.,Civ.Eng.*

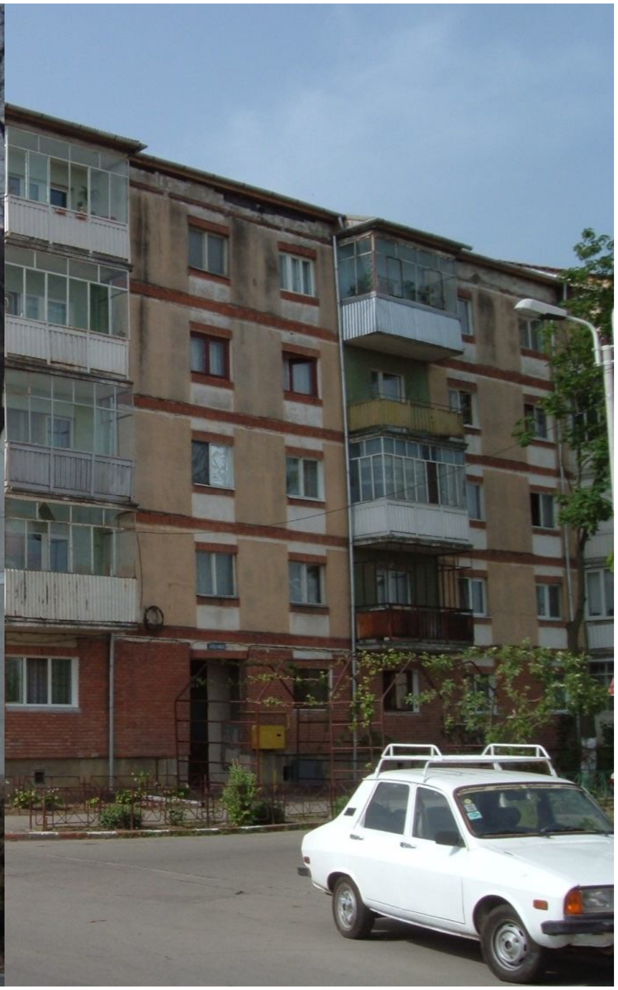
Carla TODUT *Ph.D.Stud.,Civ.Eng.*

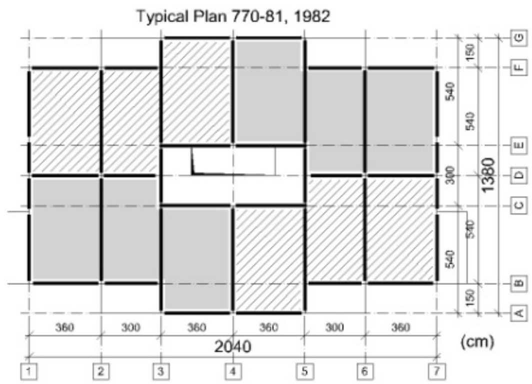


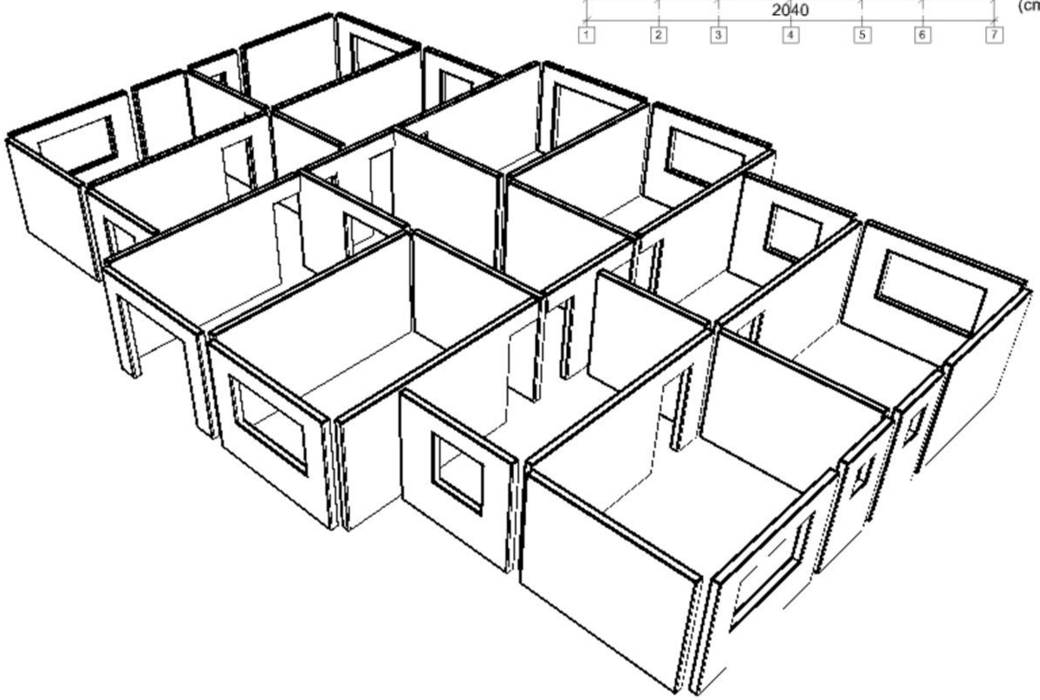




PART A – STRUCTURAL REHABILITATION USING CFRP





Architectural

Floor plan: 4 flat units and a staircase

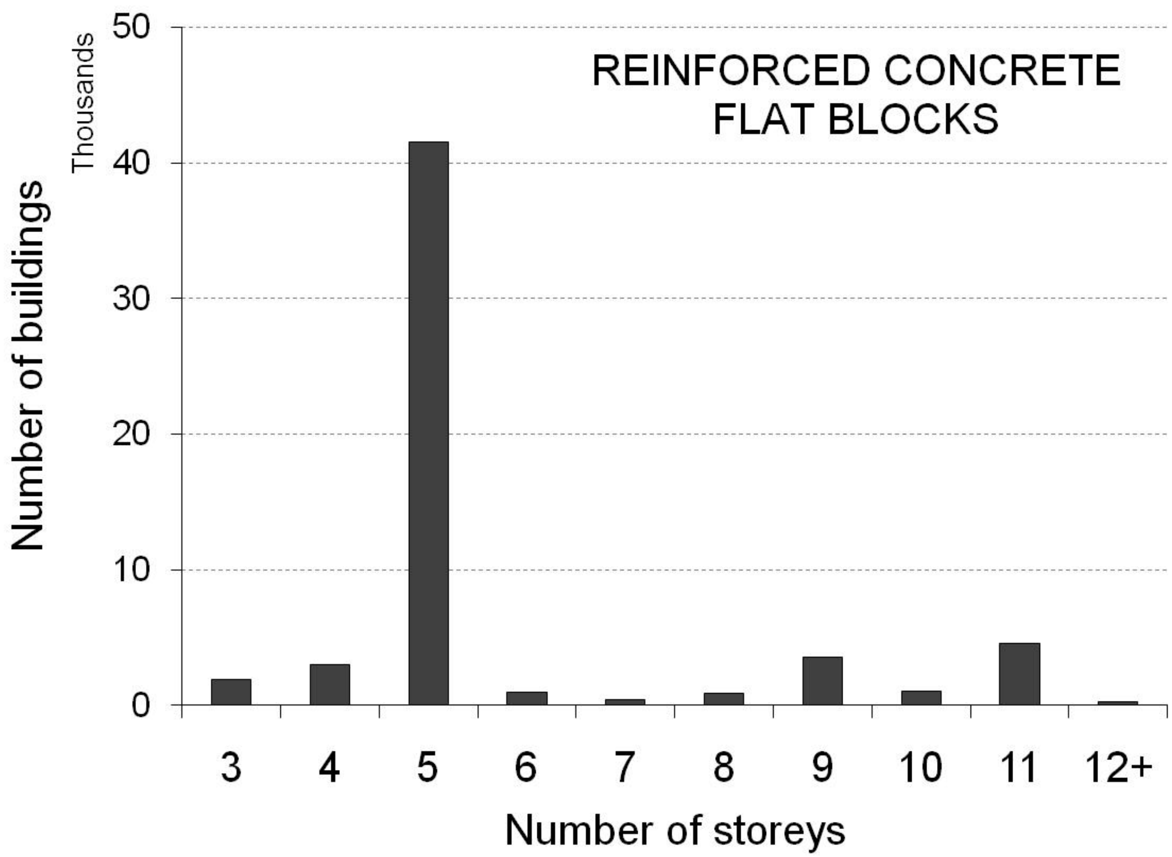
Floor area: 200÷300 sqm (total 1000÷1500 sqm)

Utility basement, Terrace roof

Structural indices

Wall to floor area (I1): 6%

Weight to wall a.(I2):0.9 MPa



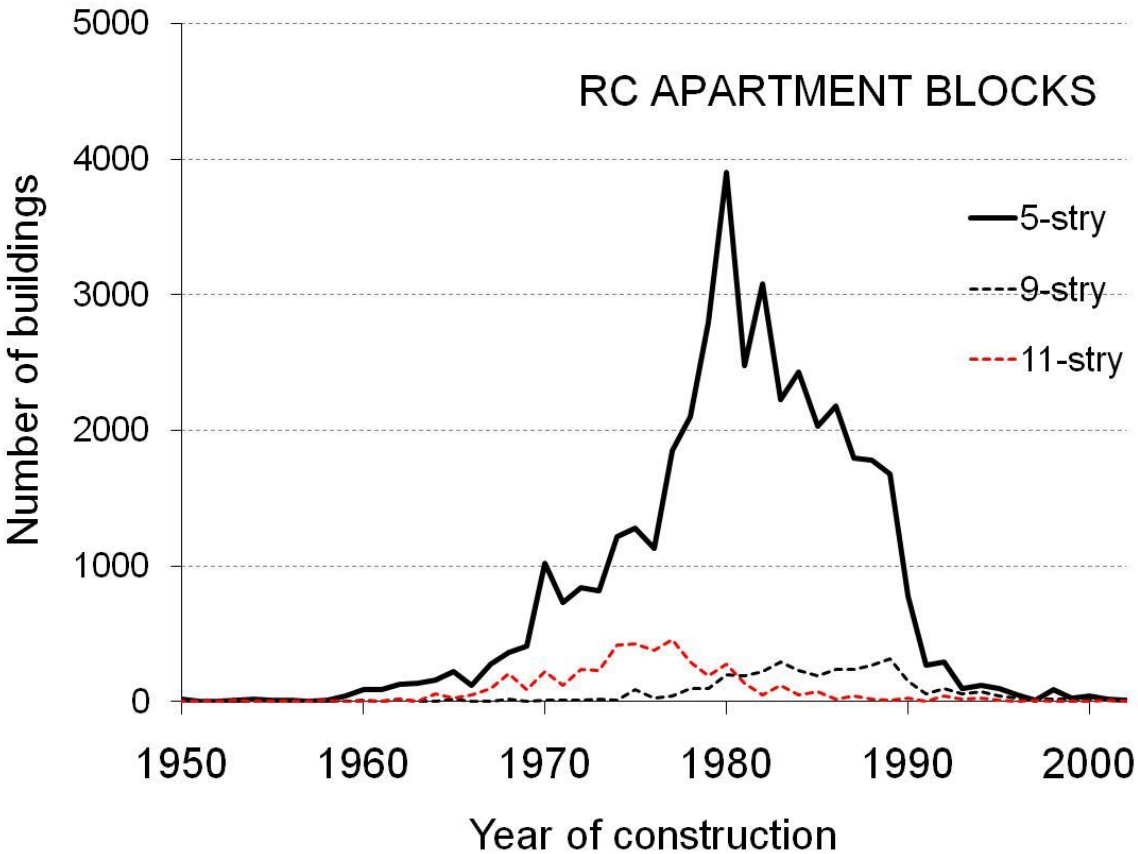
RC flat blocks

>57 000 buildings (28% of the total room area)

>40 000 are 5-storey PRCLP buildings

> 3500 are 9-storey
> 4500 are 11-storey

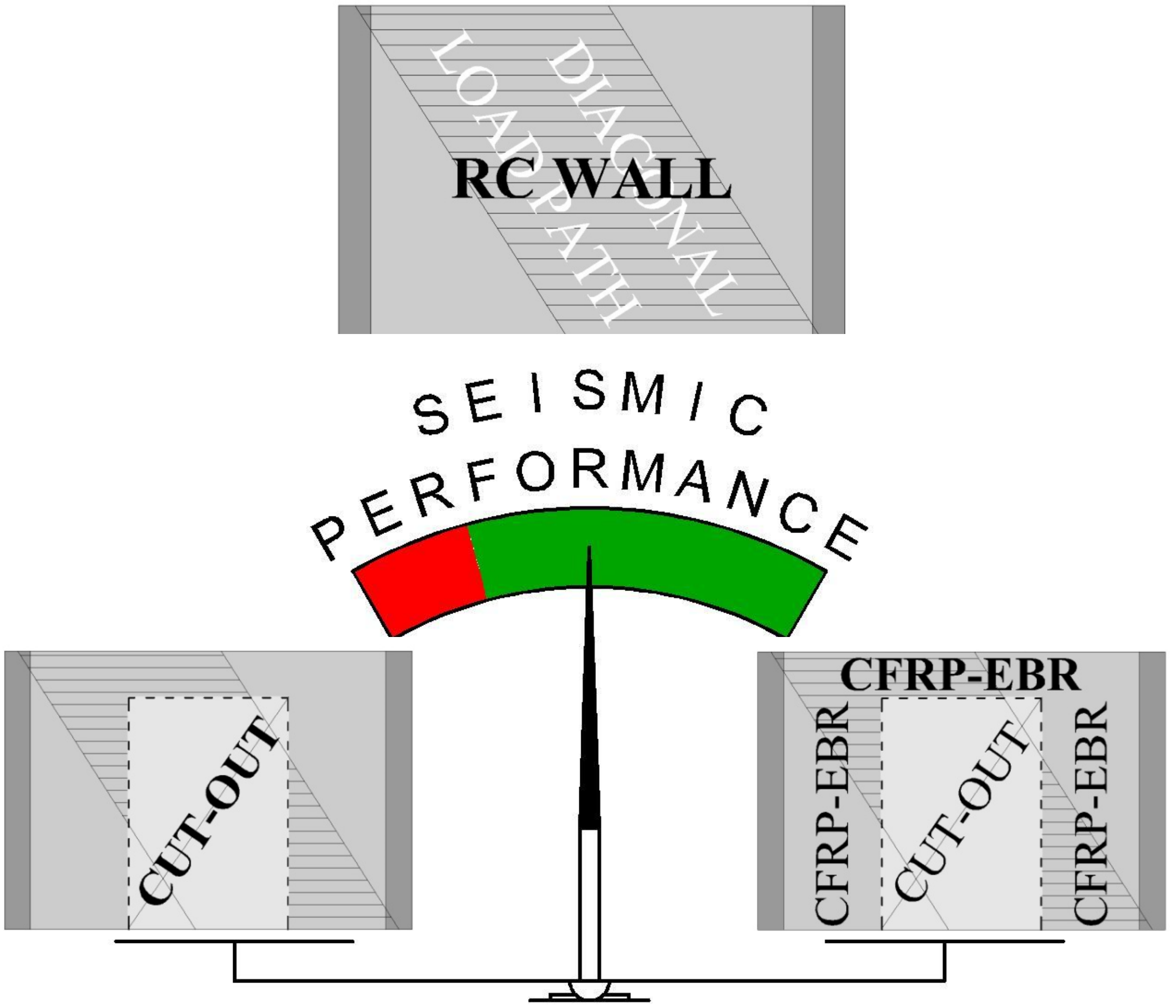
Source:
Rom. Nat. Inst. of Statistics www.insse.ro
Census of Population and Dwellings 2002



History of PRCLPs
Early period in the 1960s
Wide application started in 1970
1970-1990 the decades of 5-storey PRCLPs (**>3**6000 buildings constructed)
1990 decline

Source:
Rom. Nat. Inst. of Statistics
www.insse.ro
Census of Population and Dwellings 2002

PART A – STRUCTURAL REHABILITATION USING CFRP

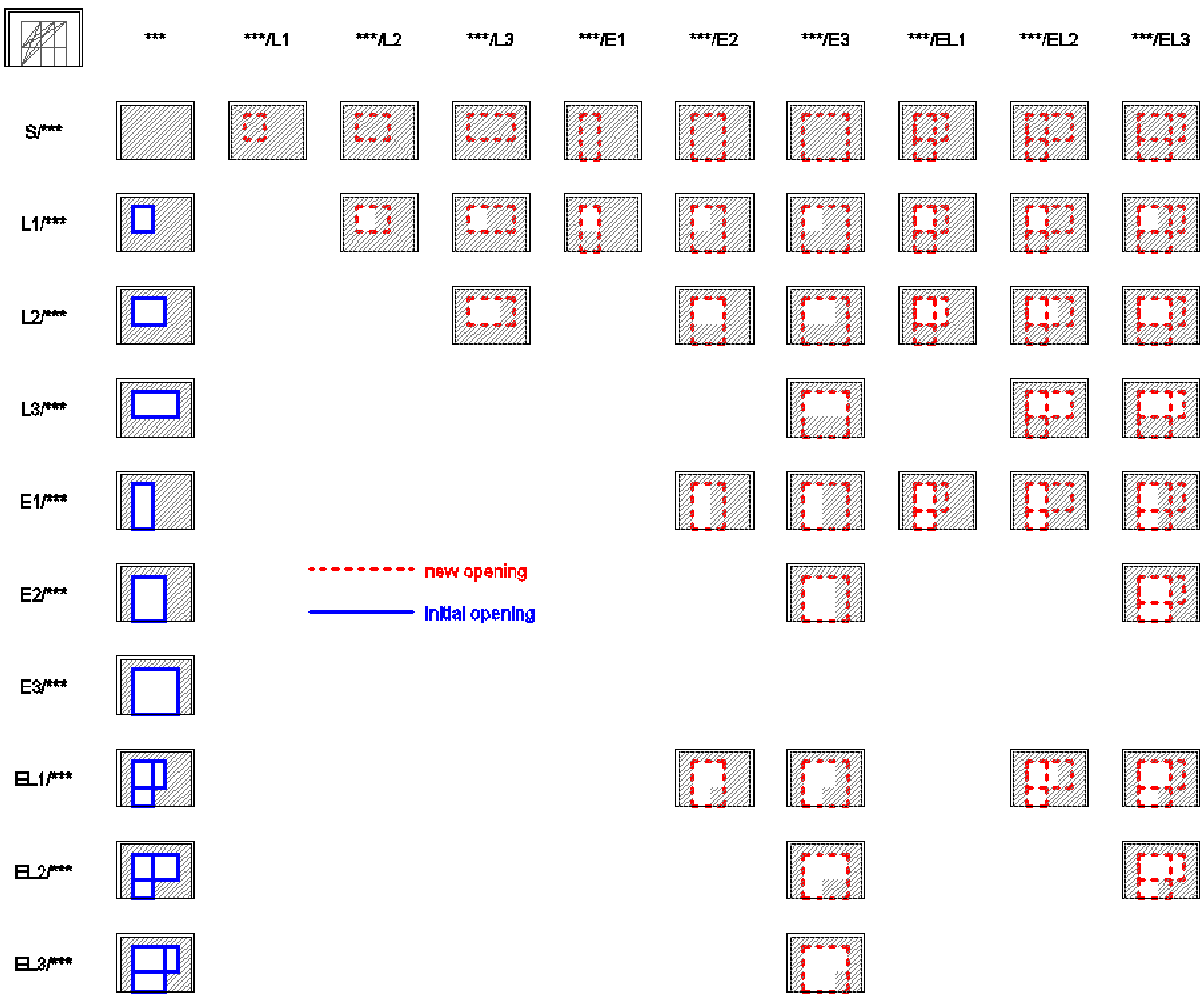


Reinforced concrete (RC) shear walls are one of the most reliable lateral load resisting systems

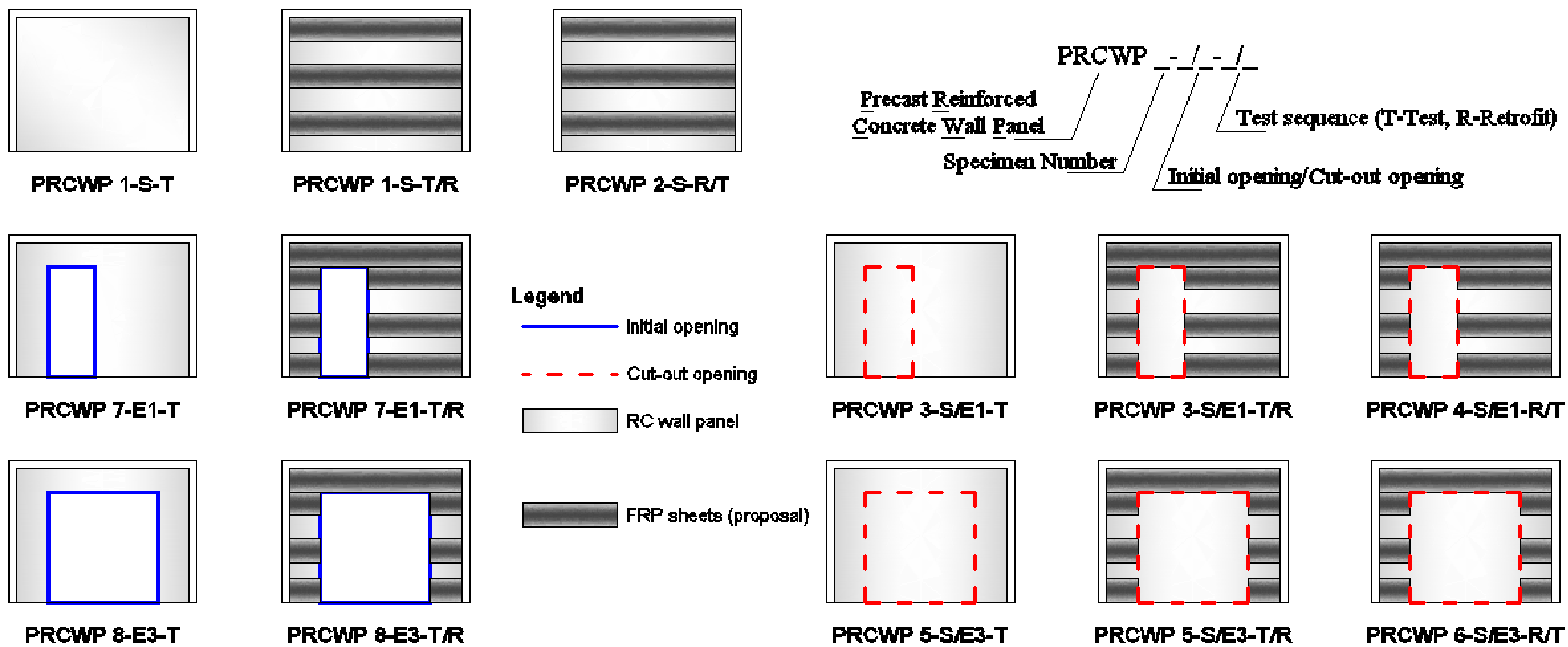
Weakening caused by cut-out openings

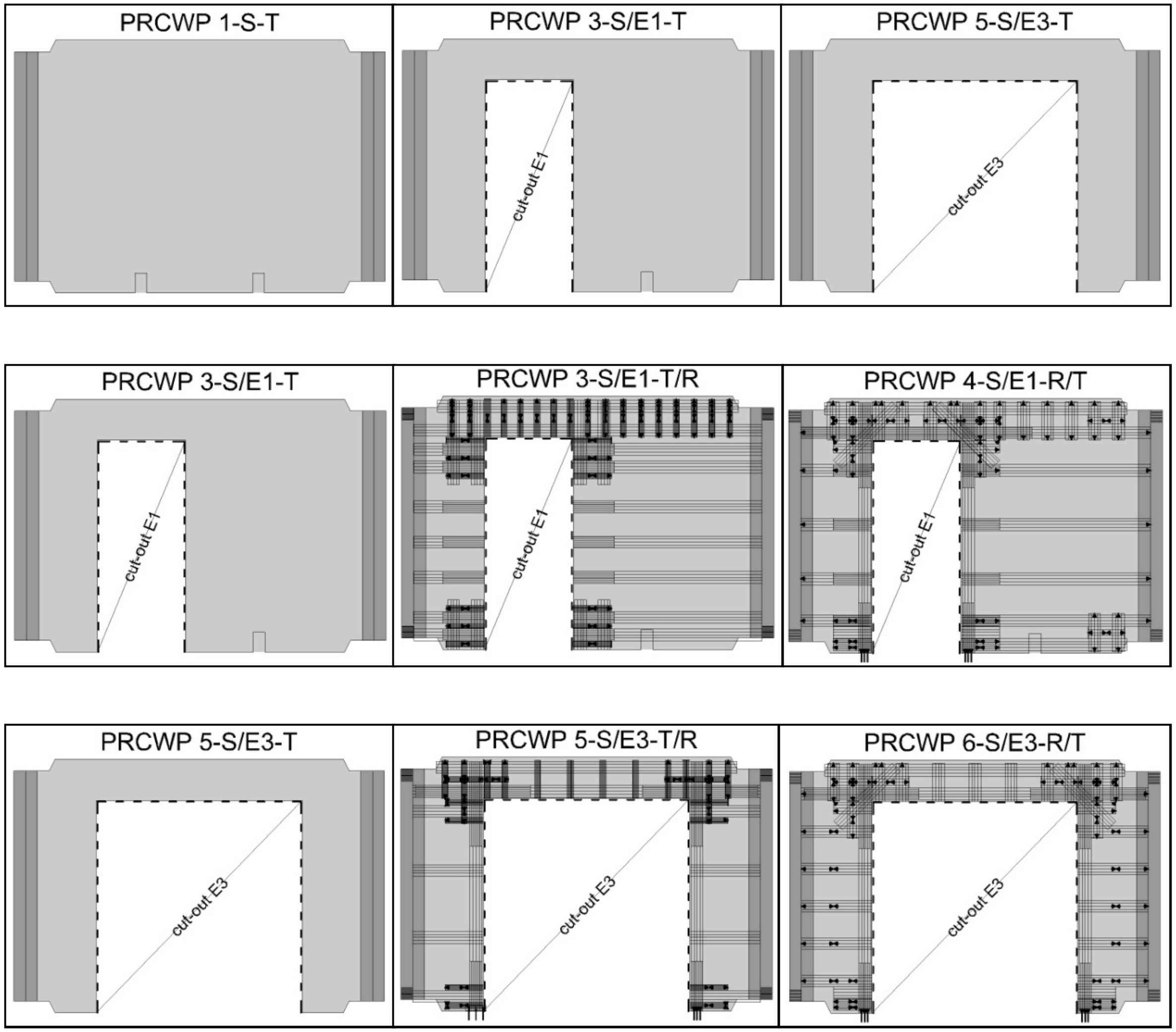
Strengthening attainable by externally bonded carbon fibre reinforced polymers (CFRP-EBR)

EXPERIMENTAL PROGRAMME *Opening configuration matrix*



EXPERIMENTAL PROGRAMME





Line 1

<u>Weakening effect of doorway cut-out</u>

REFERENCE: solid wall

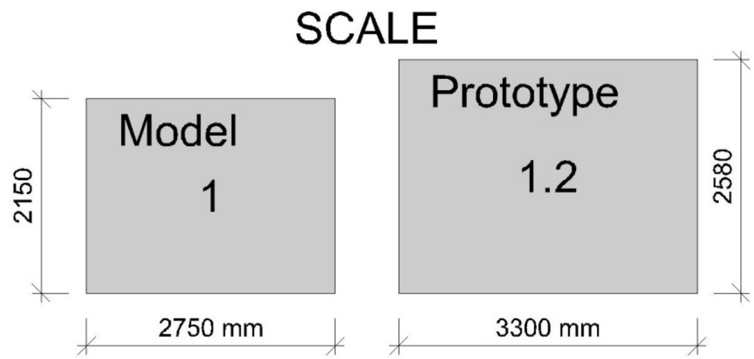
VARIABLE: cut-out width

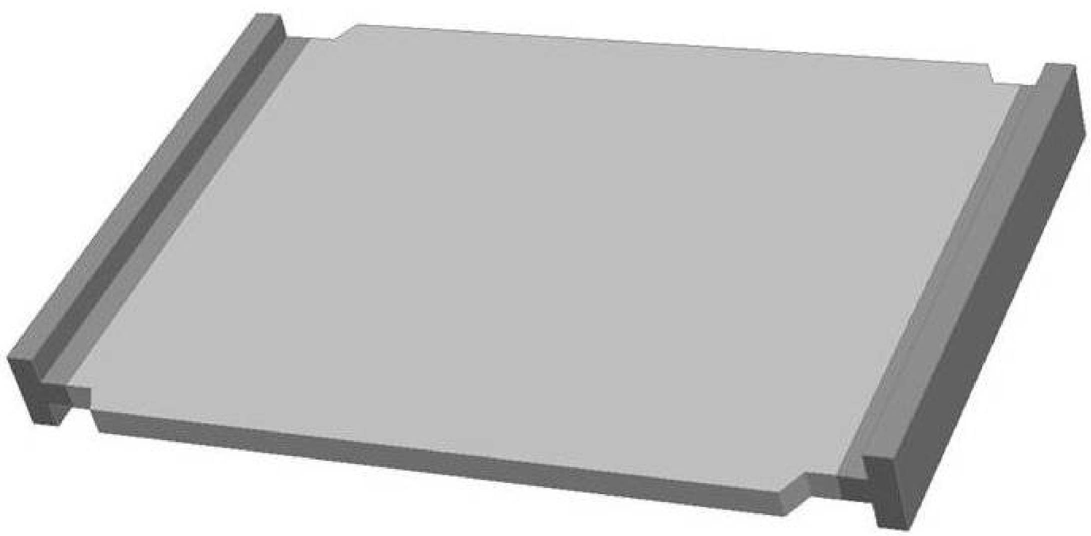
Lines 2 and 3

<u>Strengthening effect of CFRP-EBR</u>

REFERENCE: bare wall with cut-out door

VARIABLE: strengthening condition





Prototype: wall panel I 36-1, 770-81, 1982

Type: wall element (1-stry)

Scale: 0.83 (1:1.2)

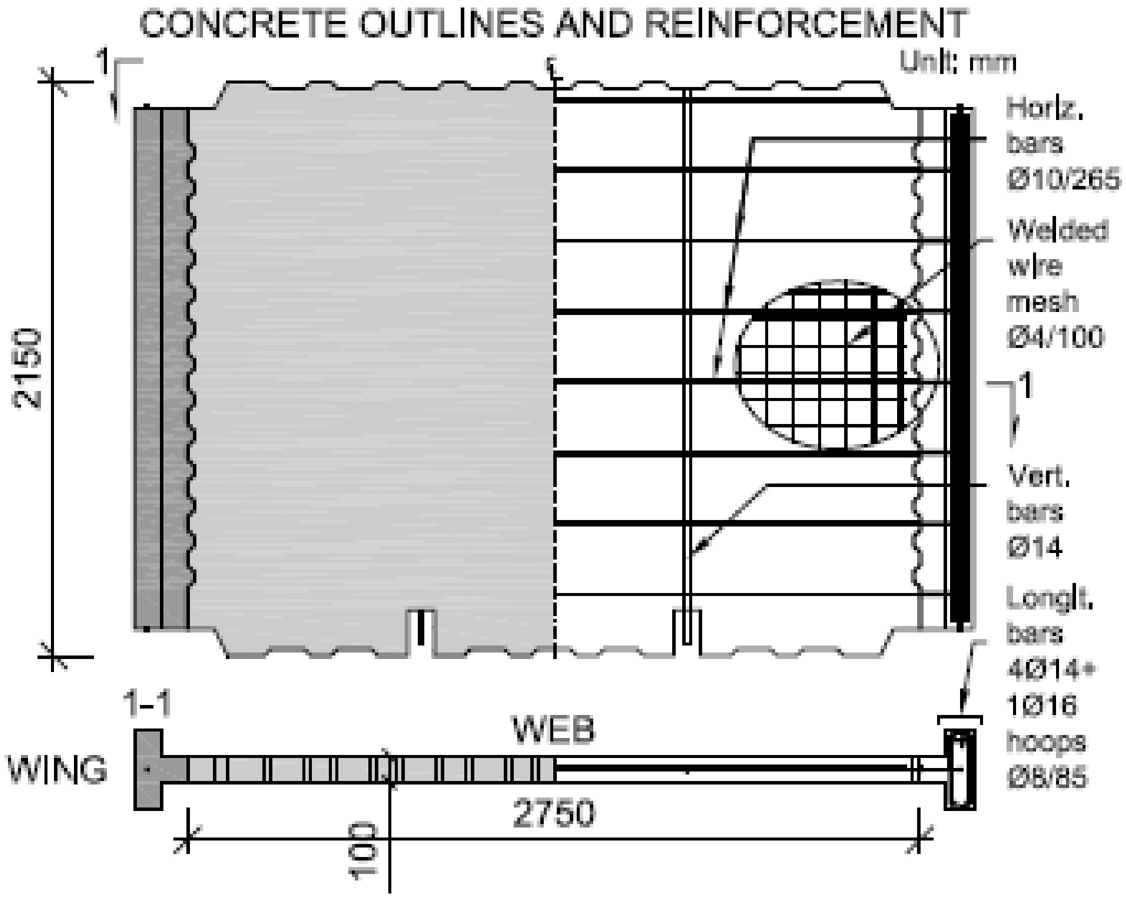
Aspect ratio: 0.8

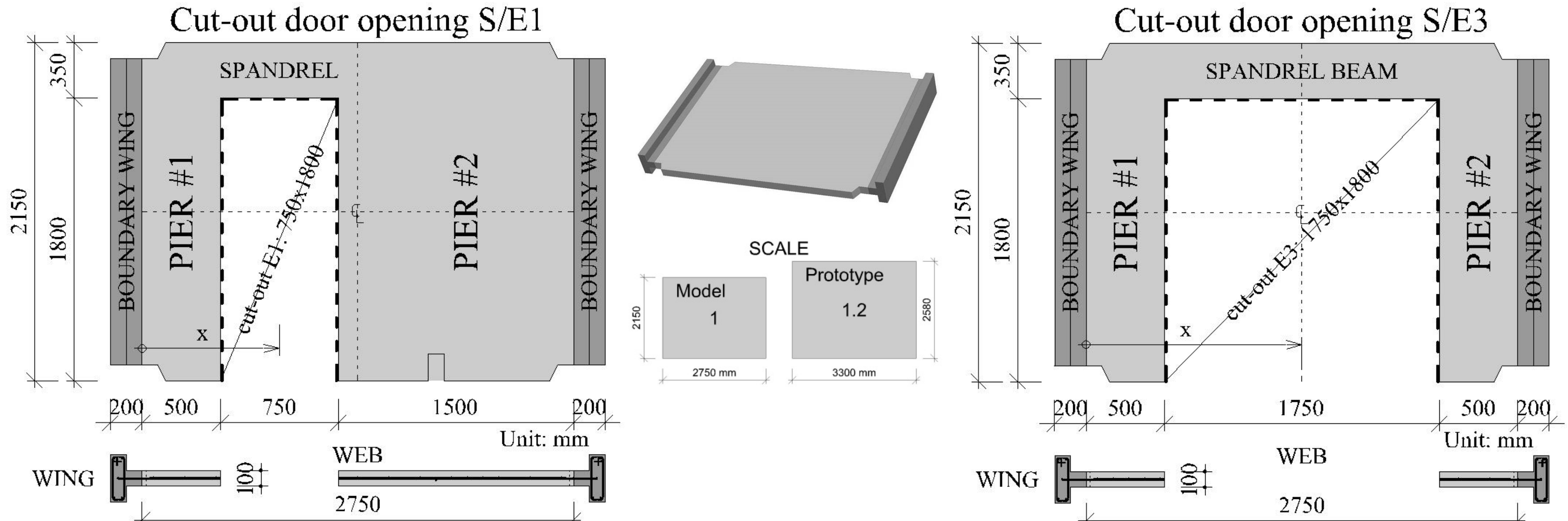
Cross section type: flanged

Components: web-panel and boundary wings

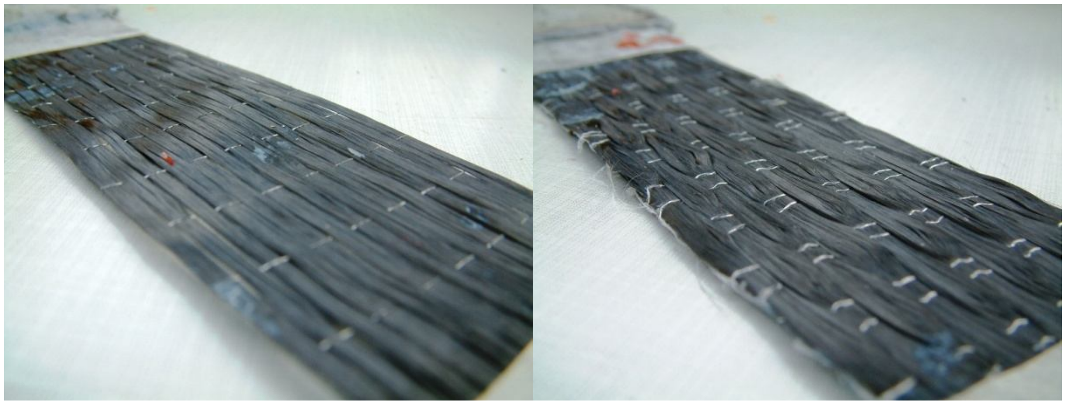
Concrete: precast

Test specimen description





Element designation	*As-built / cut-out opening type*	*Strengthening condition*
PRCWP 1-S-T	**Solid (S)**	**Bare (T)**
PRCWP 3-S/E1-T	**Solid / narrow door (S/E1)**	
PRCWP 5-S/E3-T	**Solid / wide door (S/E3)**	
PRCWP 3-S/E1-T/R	**Solid / narrow door (S/E1)**	**Post-damage strengthened (T/R)**
PRCWP 4-S/E1-R/T		**Prior-to-damage strengthened (R/T)**
PRCWP 5-S/E3-T/R	**Solid / wide door (S/E3)**	**Post-damage strengthened (T/R)**
PRCWP 6-S/E3-R/T		**Prior-to-damage strengthened (R/T)**



CFRP-EBR reinforcement

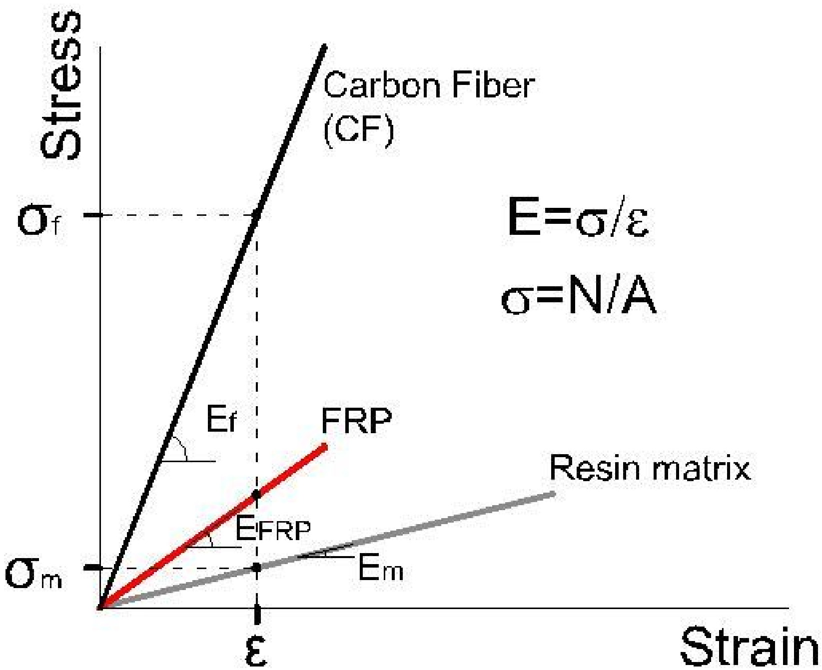
S1-type CF sheet
Unidirectional
Thickness: 0.122 mm

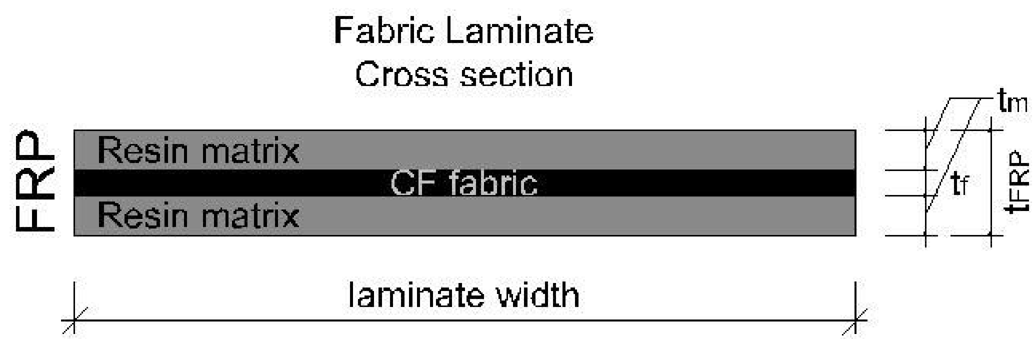
S2-type CF sheet
Unidirectional
Thickness: 0.337 mm

Impregnation resin
Tensile strength:
30÷45 MPa

Carbon Fibre (CF)	**S1 CF-sheet**	**S2 CF-sheet**
Tensile strength (MPa)	4100	3900
Tensile elongation at break (%)	1.5	1.5

CFRP-EBR reinforcement





Rule of mixtures

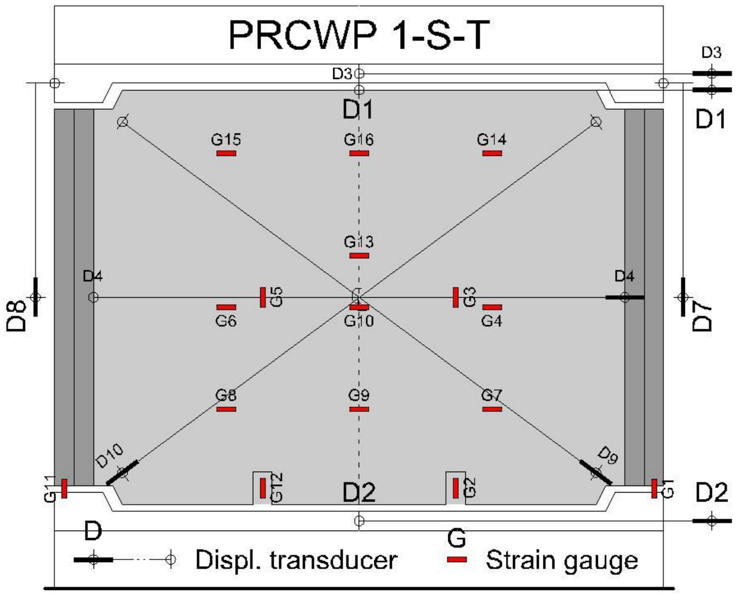
$$E_{FRP} = \frac{E_f \varepsilon A_f + E_m \varepsilon A_m}{A_{FRP}\varepsilon} = E_f \frac{A_f}{A_{FRP}} + E_m \frac{A_m}{A_{FRP}}$$

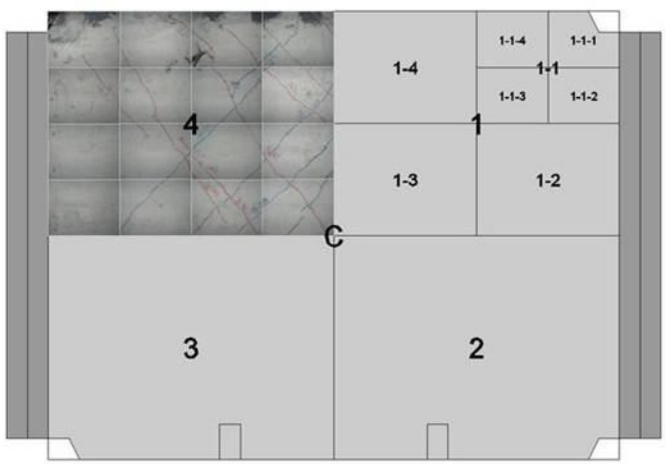
Loading degree:
4 = (2N+2V)

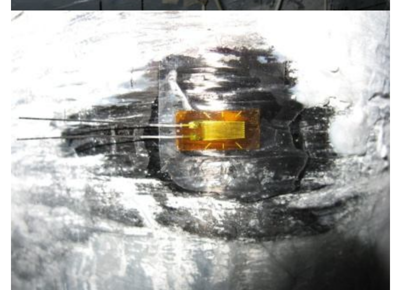
Base and cap beams: heavily reinforced steel concrete composite

Base beam not fixed, only supported

Specimen-to-base beam anchorage: lap-welding of 4 re-bars (ratio 0.17%)







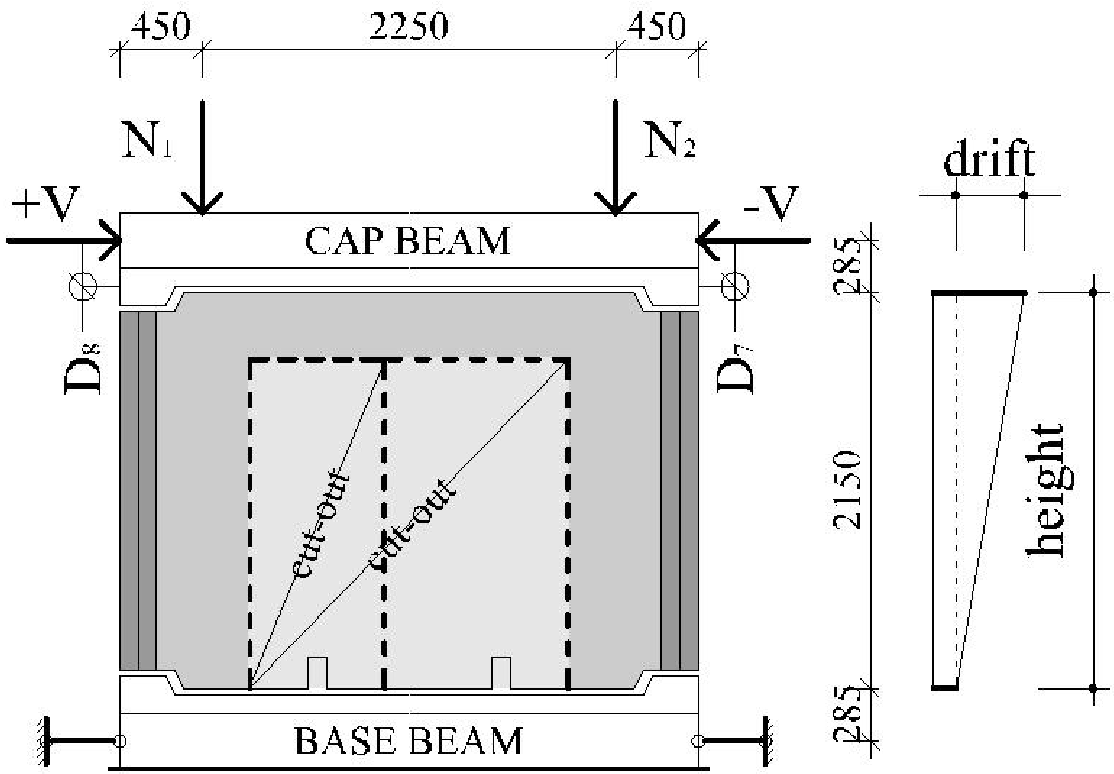
<u>Displacement</u>
10+ transducers
Arrangement: horizontal, vertical, diagonal
Support: external, internal

<u>Strain gauges</u>
16 gauges
Steel bars and CFRP

<u>Load</u>
3 pressure transducers on the hydraulic lines

<u>Observation grid lines</u>

PART A – STRUCTURAL REHABILITATION USING CFRP



Drift ratio

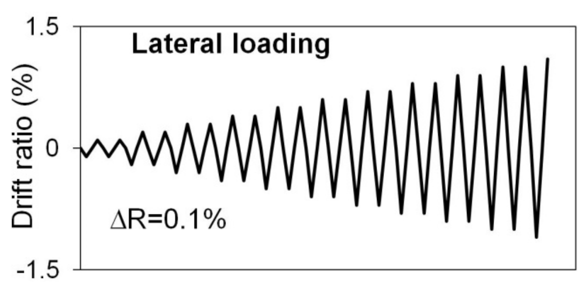
$$R\% = \frac{drift}{height} 100$$

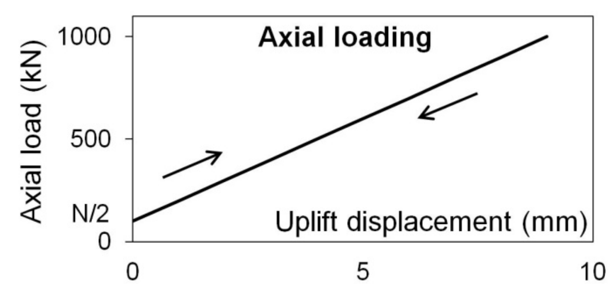
$$N = 0.06\, A_c\, f_{ck}$$

$$N_1 = N/2 + 100 D_8$$

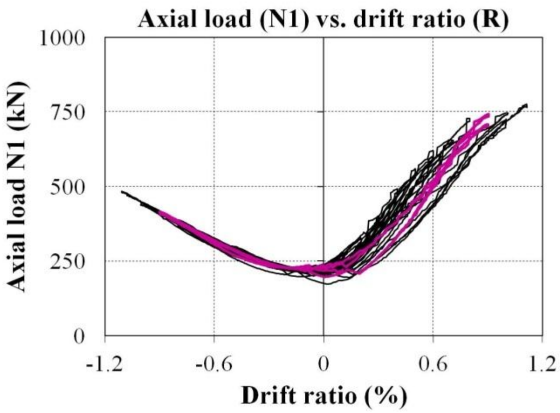
$$N_2 = N/2 + 100 D_7$$

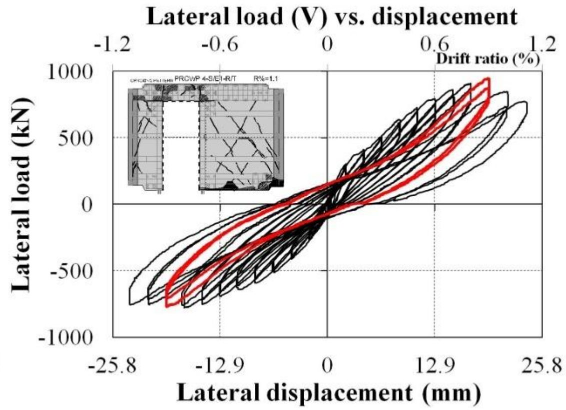
Note: N, N_1 and N_2 in kN
D_7 and D_8 in mm
Length units: mm.

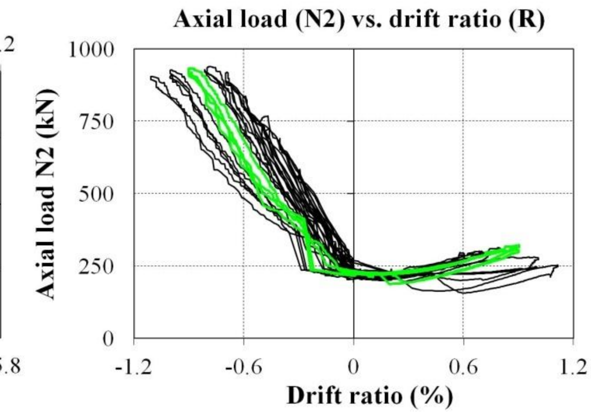


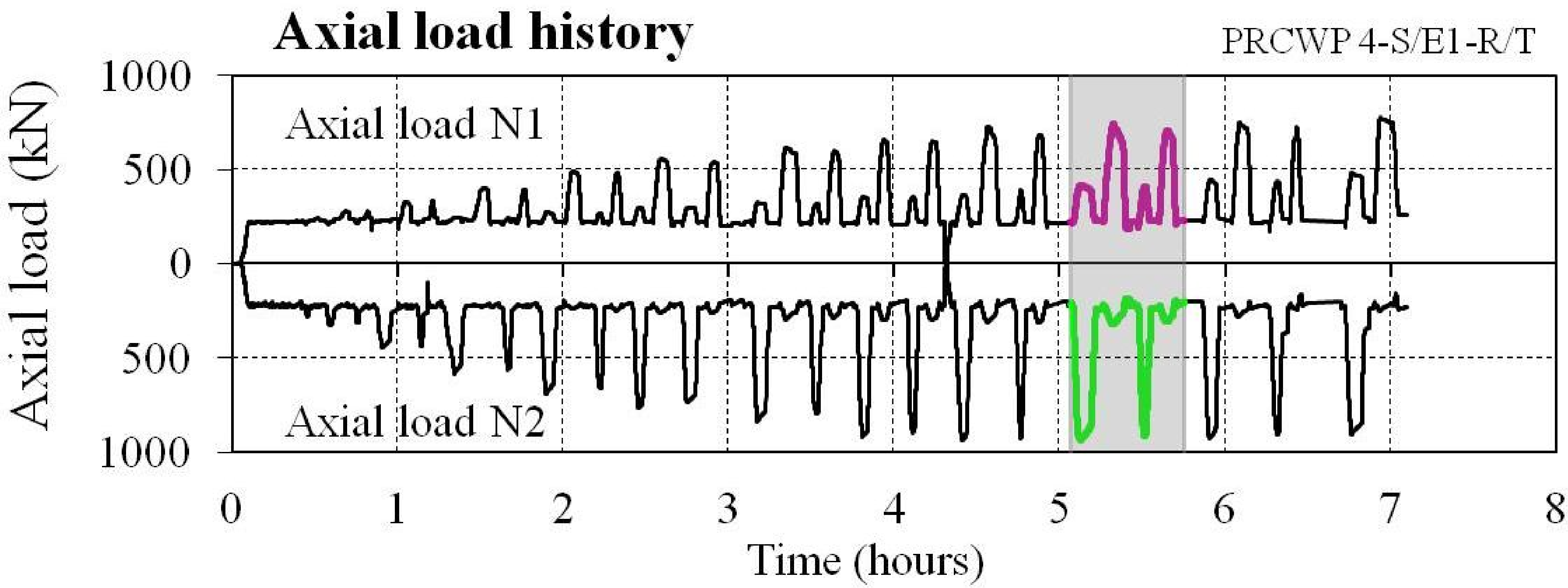


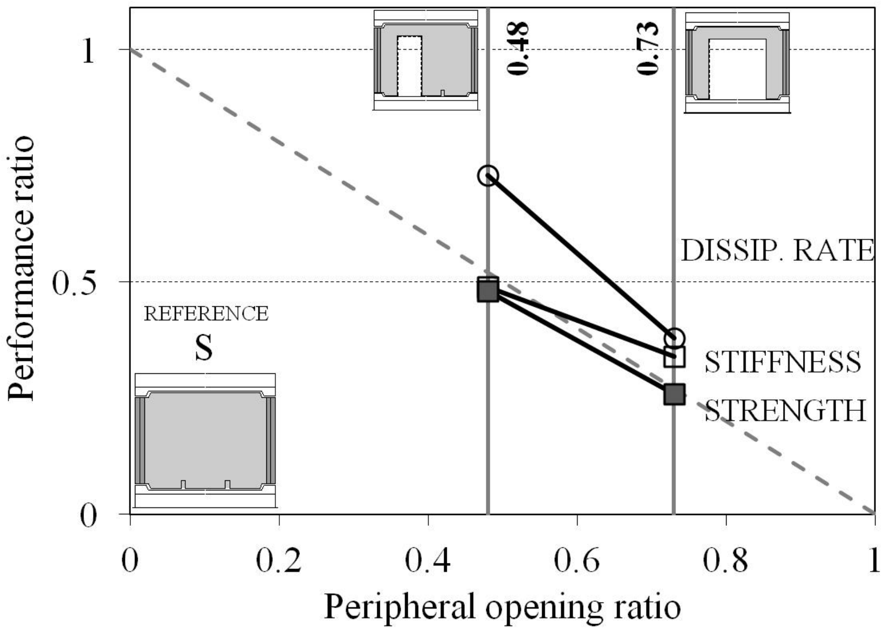
PART A – STRUCTURAL REHABILITATION USING CFRP

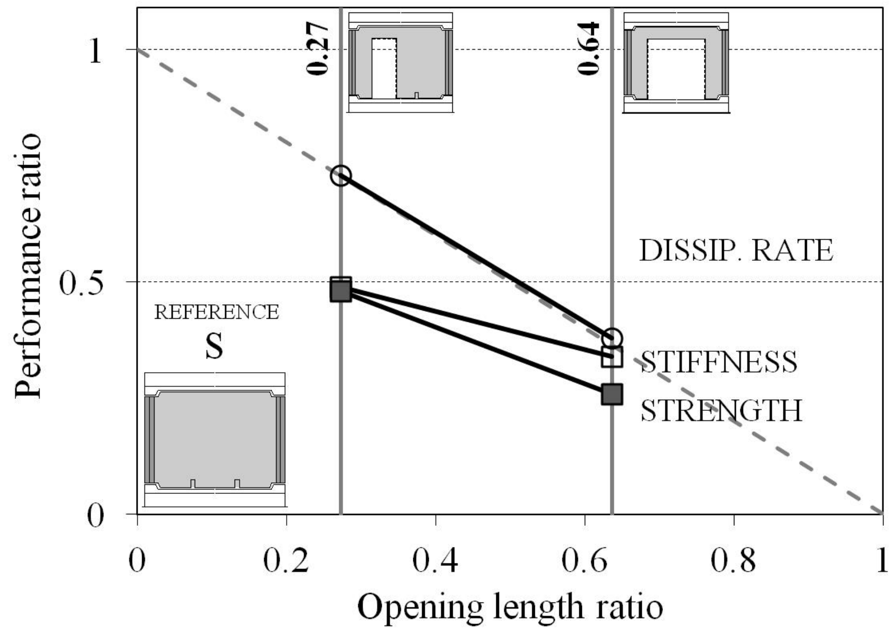










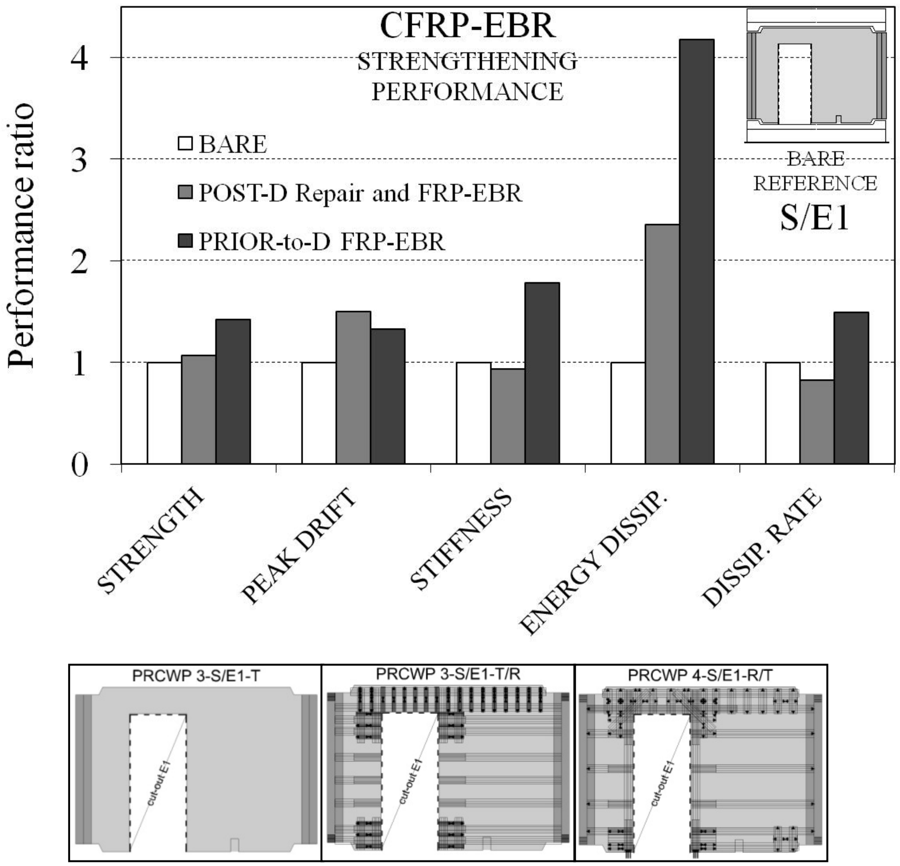


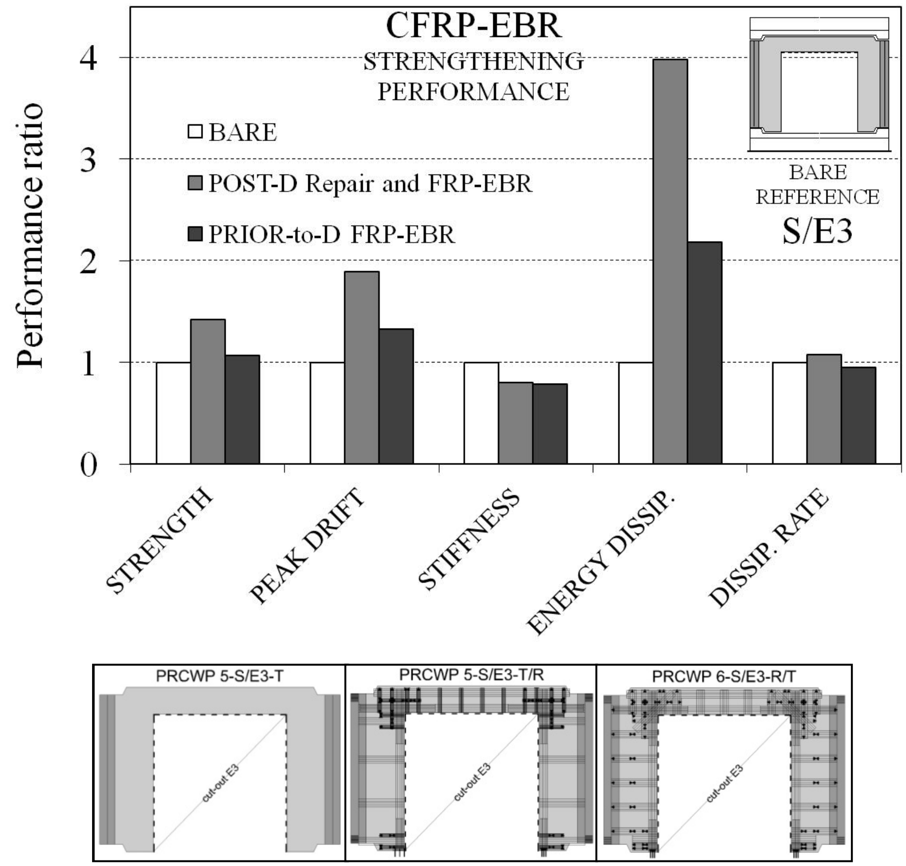
$(R)_{weak}=(R)_{sound}\alpha_p$; $\alpha_p=1-\eta$, (after AIJ 1999)

where:

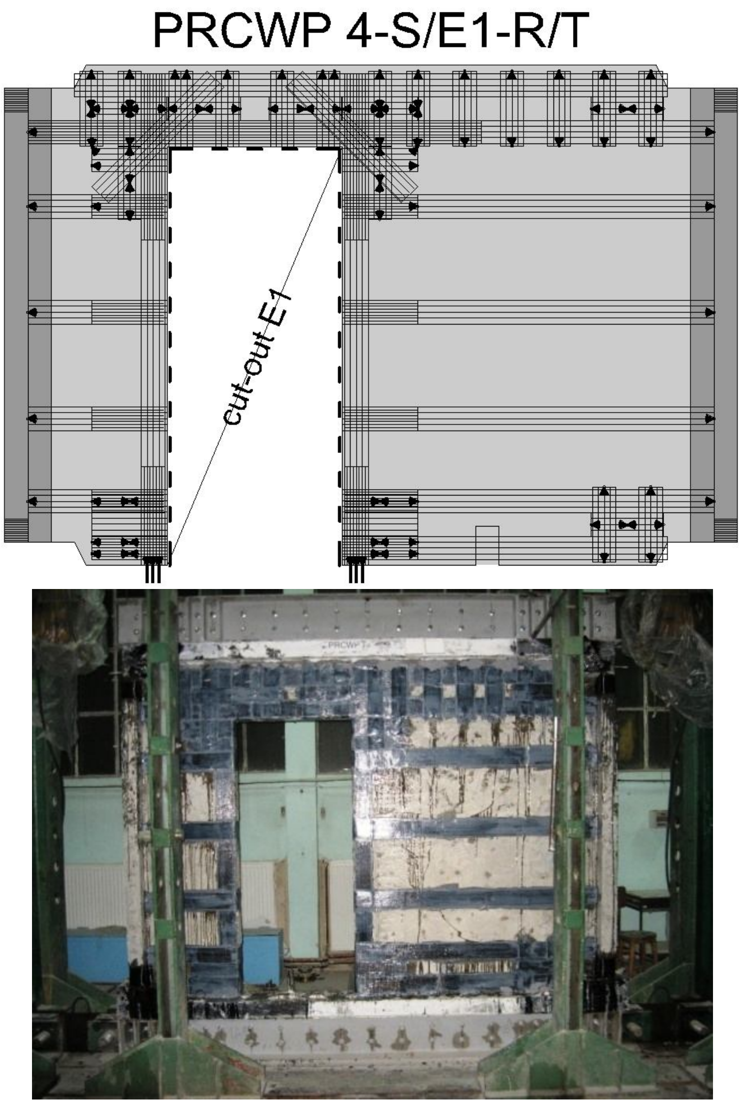
$\eta=(A_o/A_w)^{0.5}$ for shear resist. and stiffness

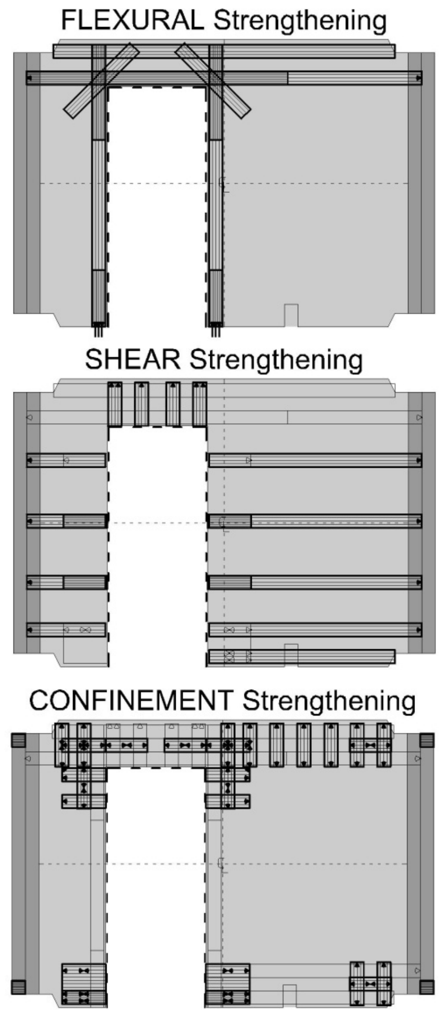
$\eta=l_o/l_w$ for energy dissipation rate





Outstanding improvement was achieved in terms of energy dissipation.





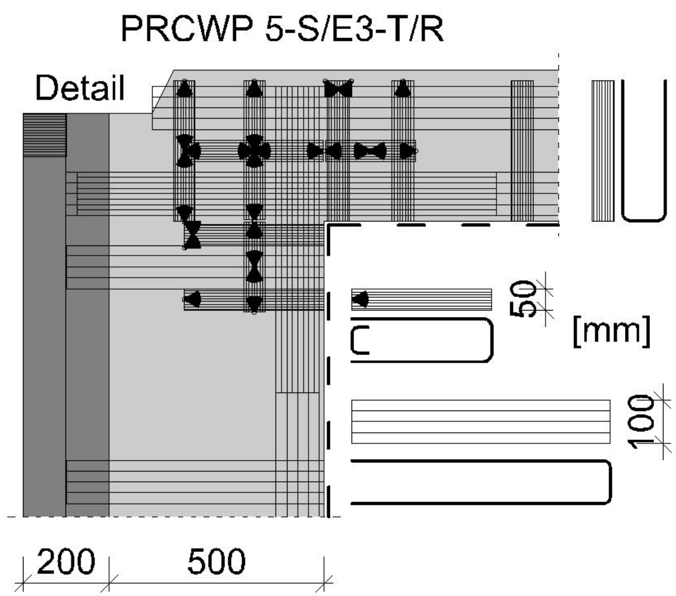
CFRP-EBR

CF-strips of 50/100 mm width

Average CFRP usage (4RT):
CF 0.85
Resin 1.2 kg/sqm

Arrangement: FL, SH, CNF

Note inclined diagonal strips at the upper corners
Improvements: end anchorage of SH-strips



Pier-beam connection

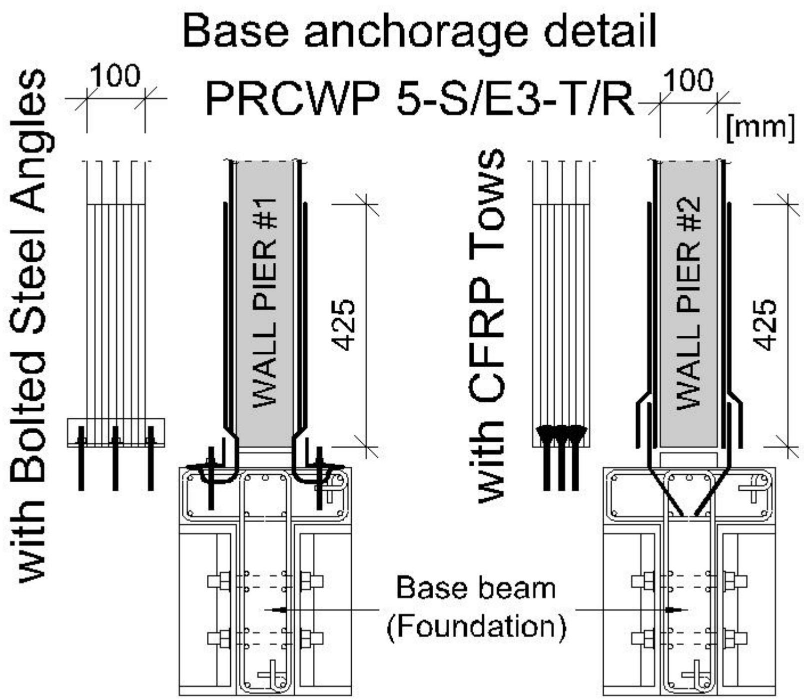
Substrate preparation

Flexural strips

Through-wall anchorages (CFRP tows)

Shear strips

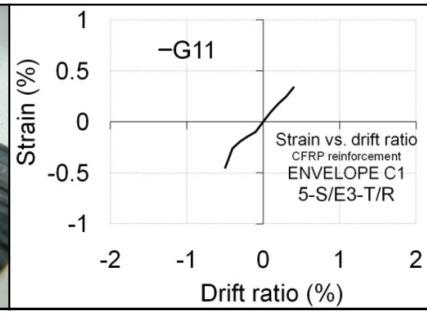
Confinement strips

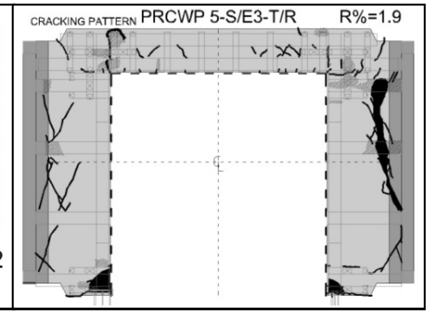


Base anchorage

<u>Solution 1</u>
Bolted steel angles

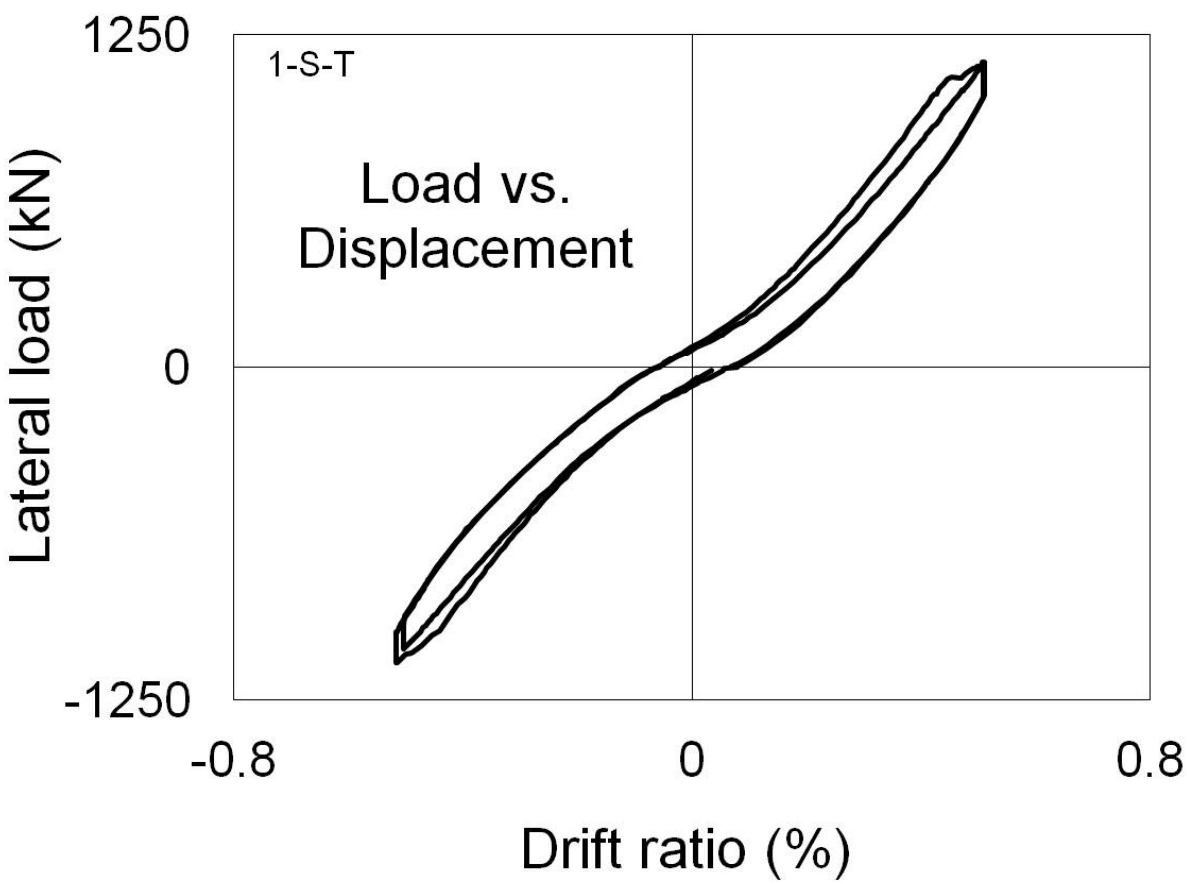
<u>Solution 2</u>
CFRP tows





CFRP-strips subjected to alternating tension-compression reversals parallel to fiber direction are likely to fail prematurely.

Further subject-oriented investigations are necessary on this issue.

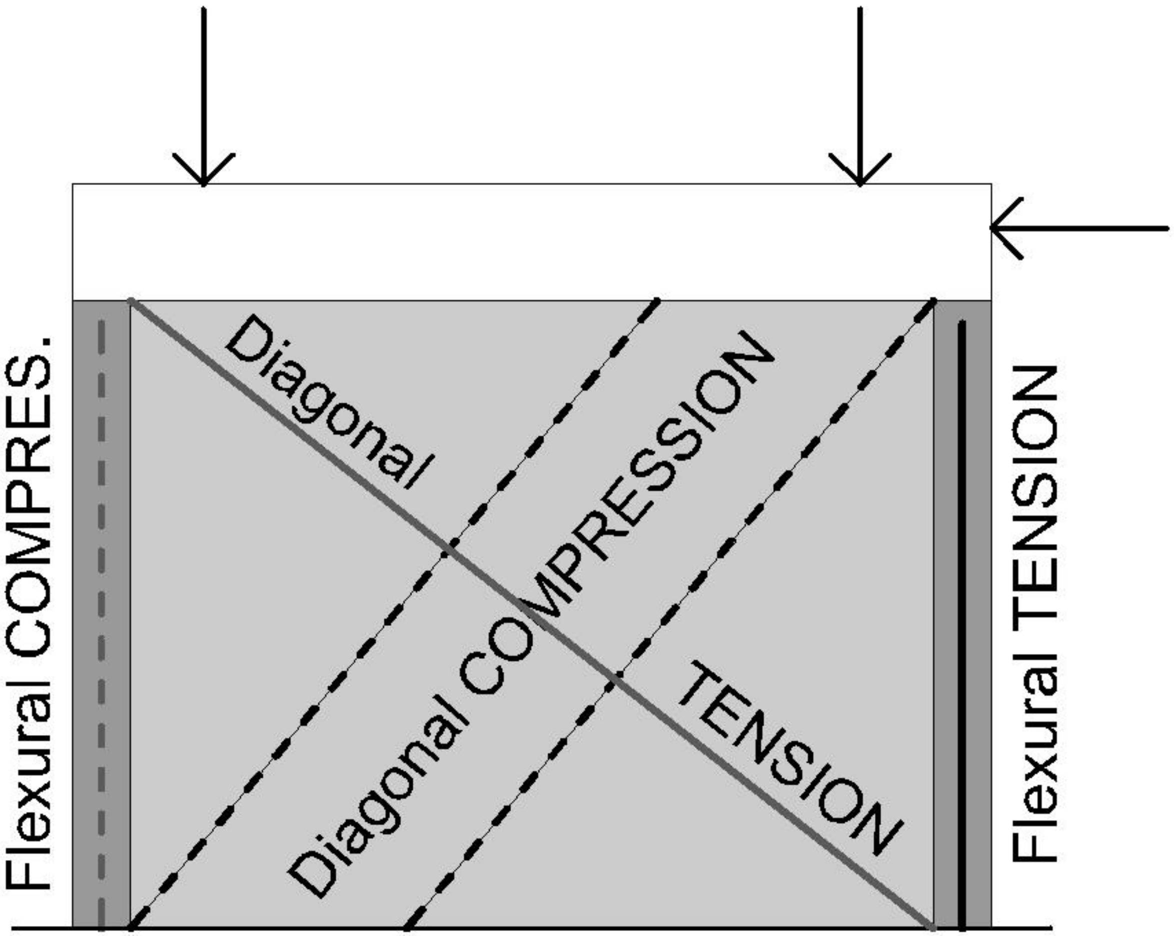


Atypical lab behaviour of RC walls

Unexpectedly high shear resistance, extremely pinched load-displ. hysteresis loops

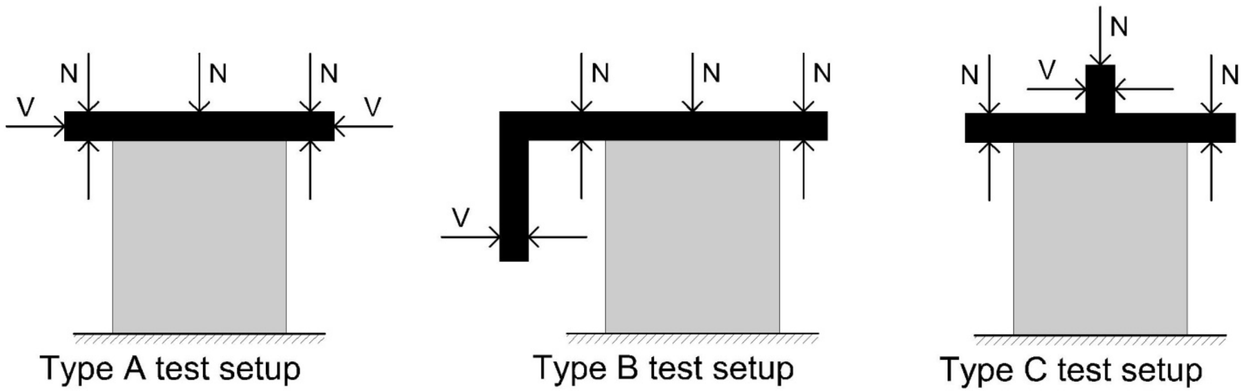
Response analysis

Strength, displacement, stiffness, energy dissipation



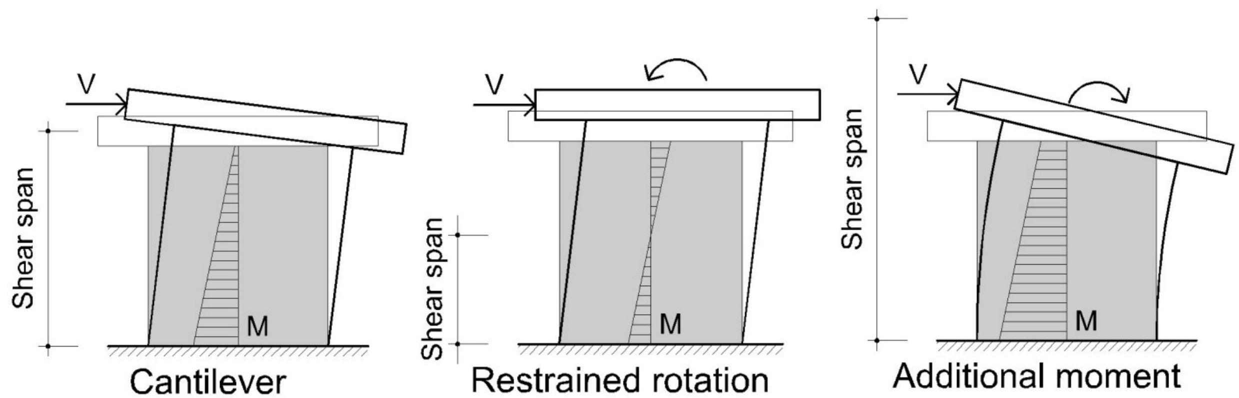
Shear transferred along diagonal load paths: DIAGONAL COMPRESSION ***and/or*** DIAGONAL TENSION

Proportion between shears carried by the two load paths:
stiffness
loading conditions
boundary conditions



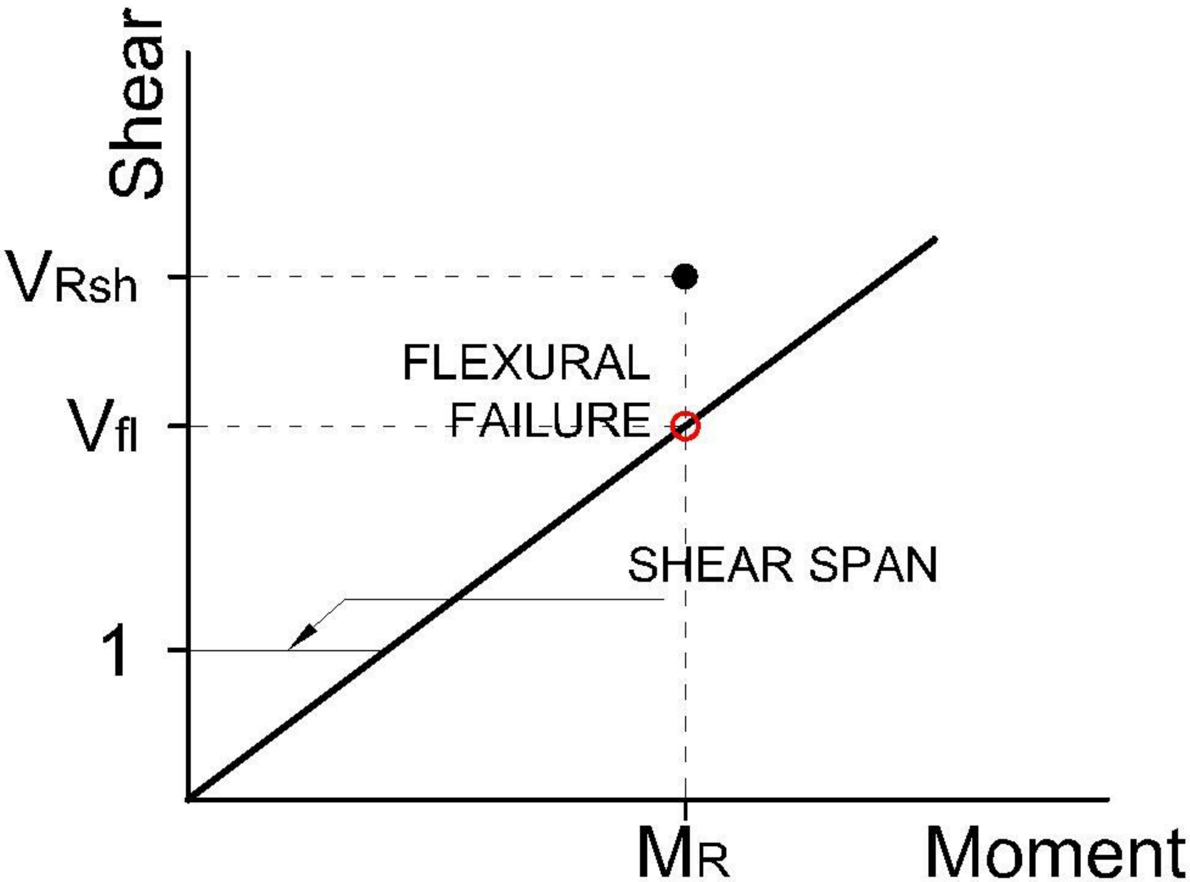
Loading degree

Number and location of the axial and lateral loads



Boundary conditions

Cantilever
Restrained rotation
Additional moment



Flexural failure

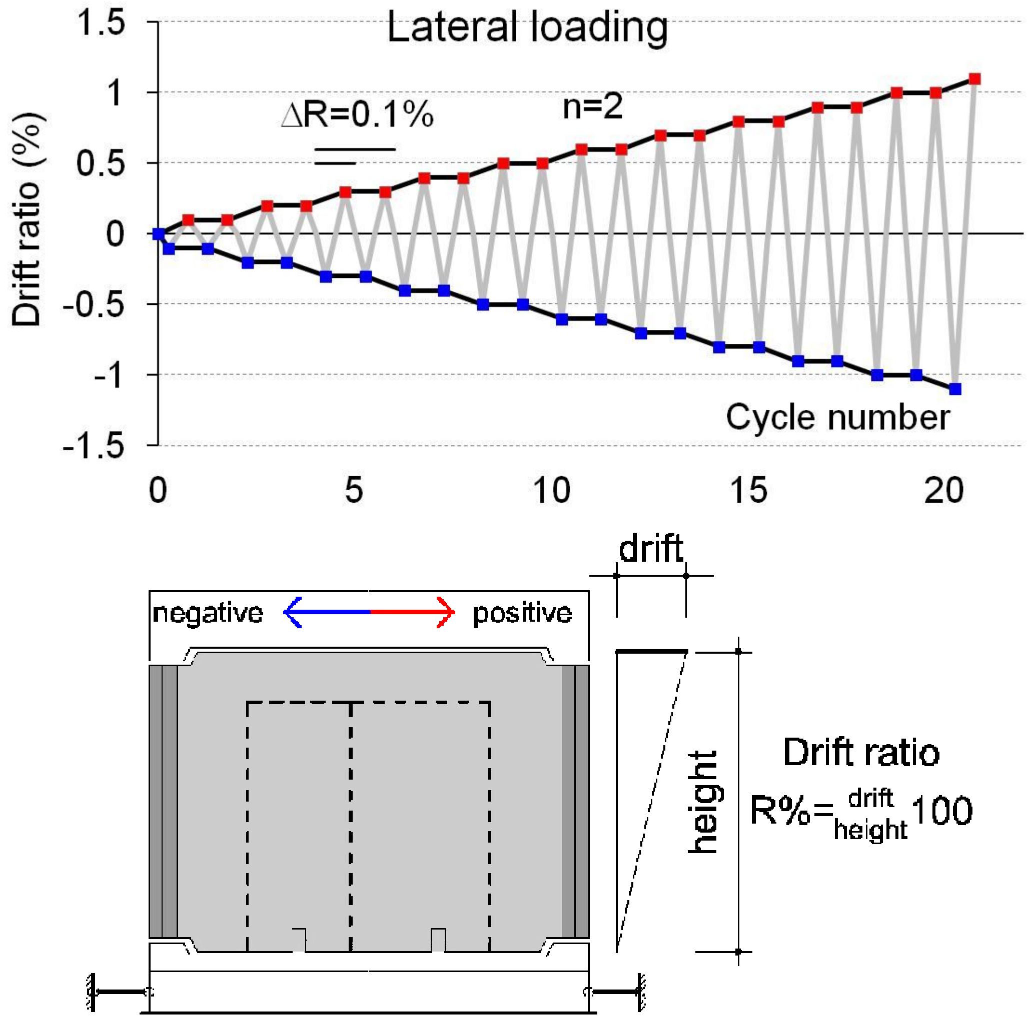
Ductile, large hysteretic loops, significant energy dissipation

Shear failure

Brittle, pinched hysteretic loops, reduced energy dissipation

FAILURE MODE CONTROLLED BY

Shear span ratio
Shear to flexural strength ratio



Lateral loading

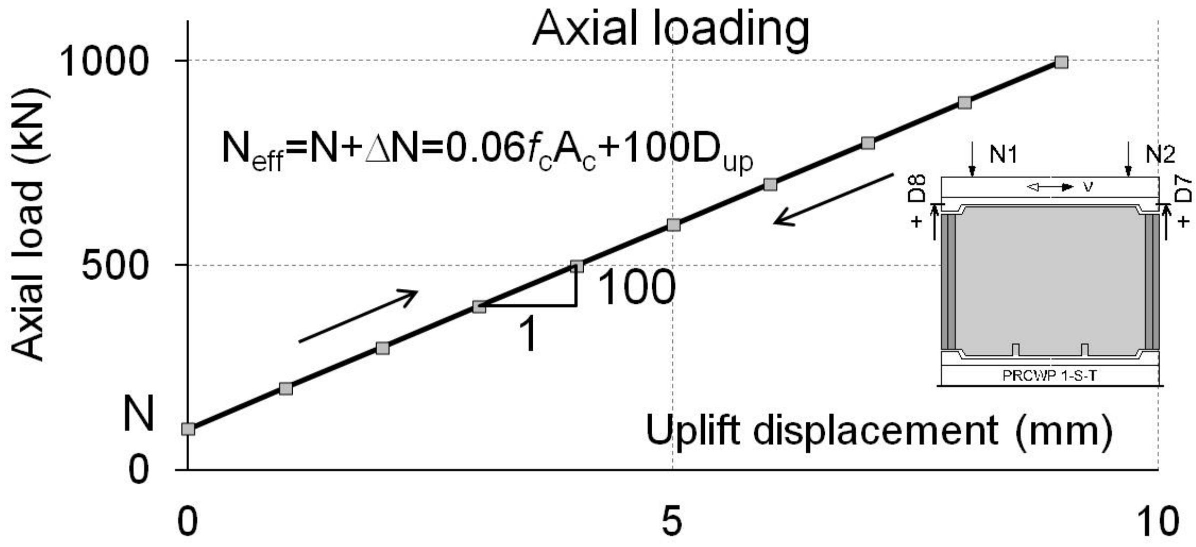
Principal characteristics: quasi-static, in-plane, reversed cyclic

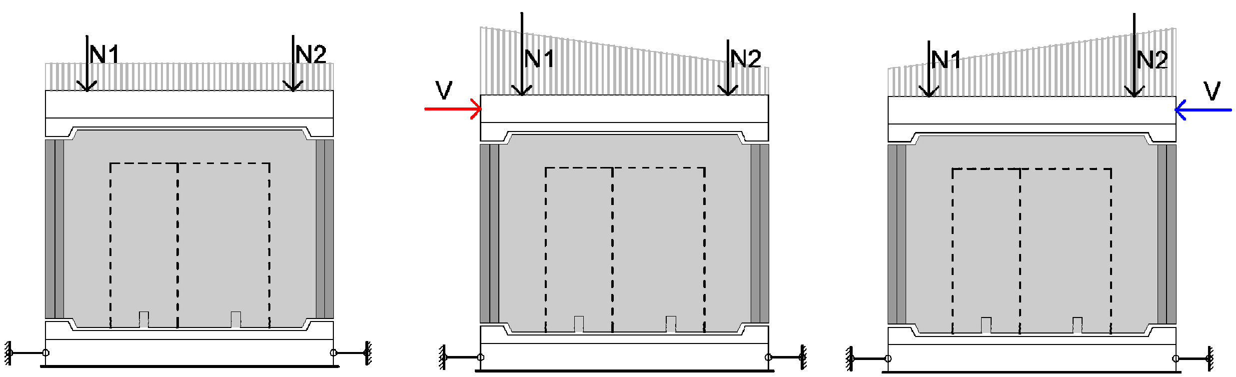
Control: drift ratio

Drift ratio increment: constant, 0.1%

Number of cycles on a level: 2

Failure criterion: 20% load carrying capacity loss





Axial loading

<u>Constant axial load level</u>

Normalised axial load: 6%

<u>Alternating component</u>

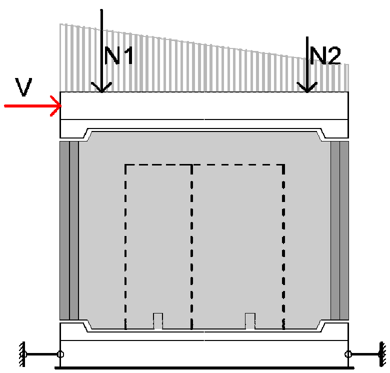
Control: displacement (uplift) of the cap beam's loaded end

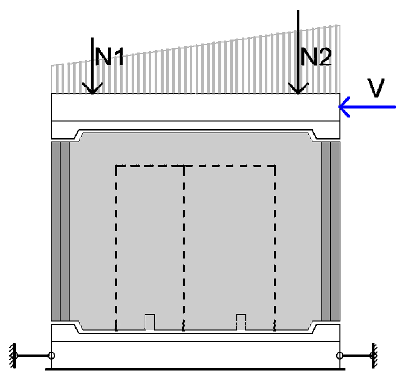
Rate: 100 kN/mm (based primarily on test-setup limitations)

Note that the base beam is not fixed to the laboratory floor

Outrigger canoe

source http://www.ballinaoutriggers.com.au

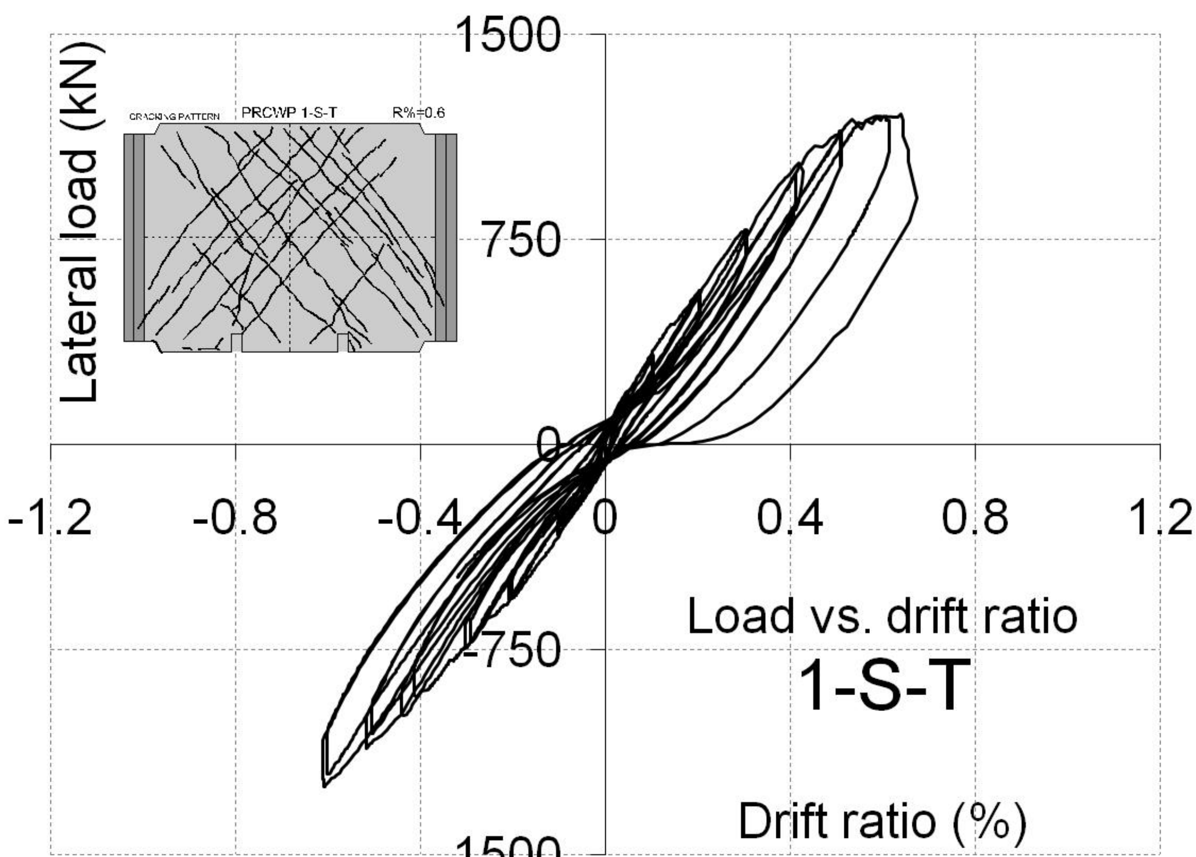


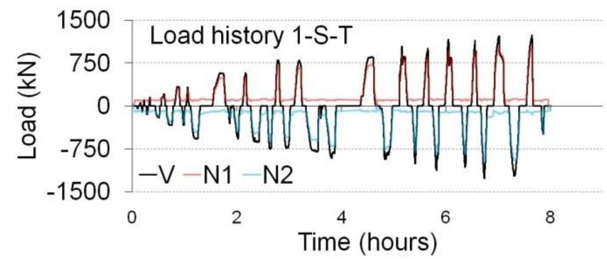


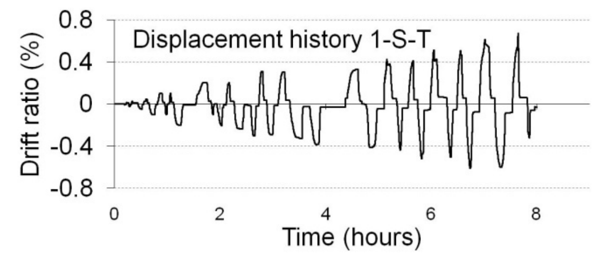
Outrigger effect

Restrained rotation by additional eccentric axial loading

PART A – STRUCTURAL REHABILITATION USING CFRP







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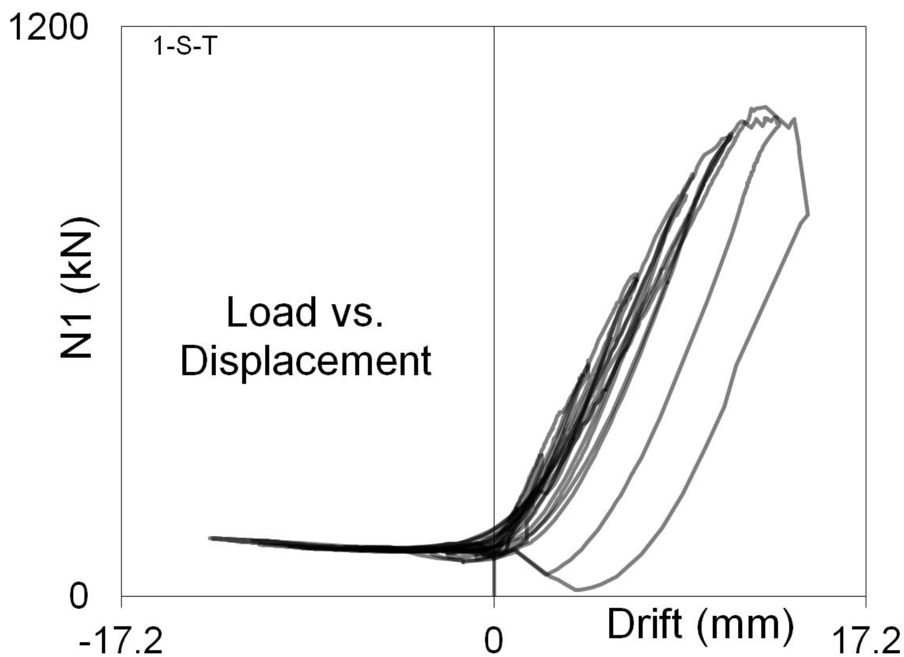
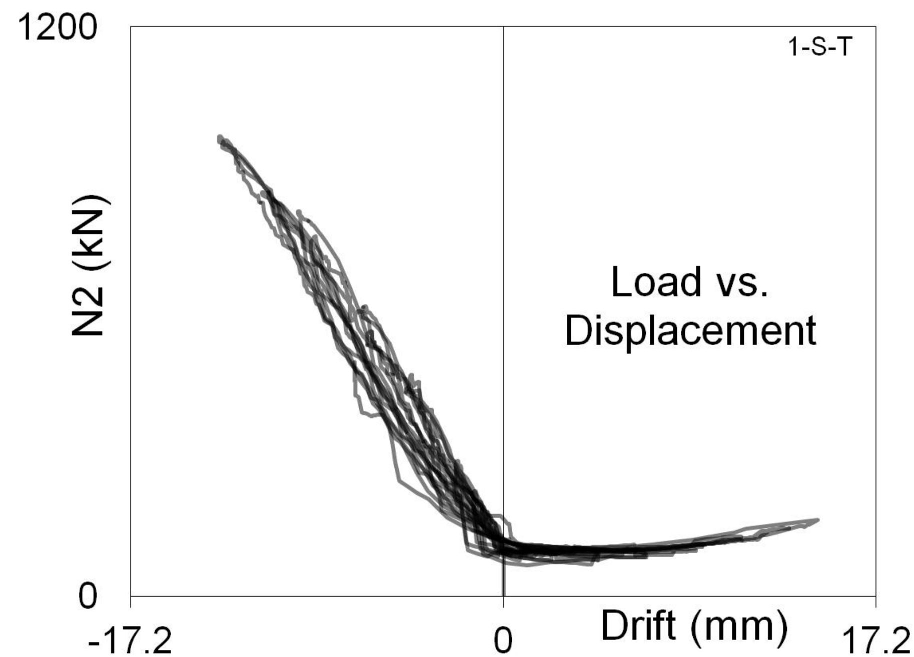
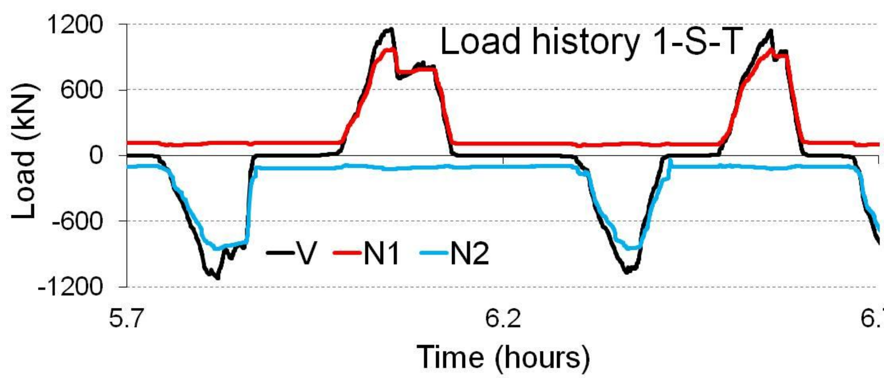
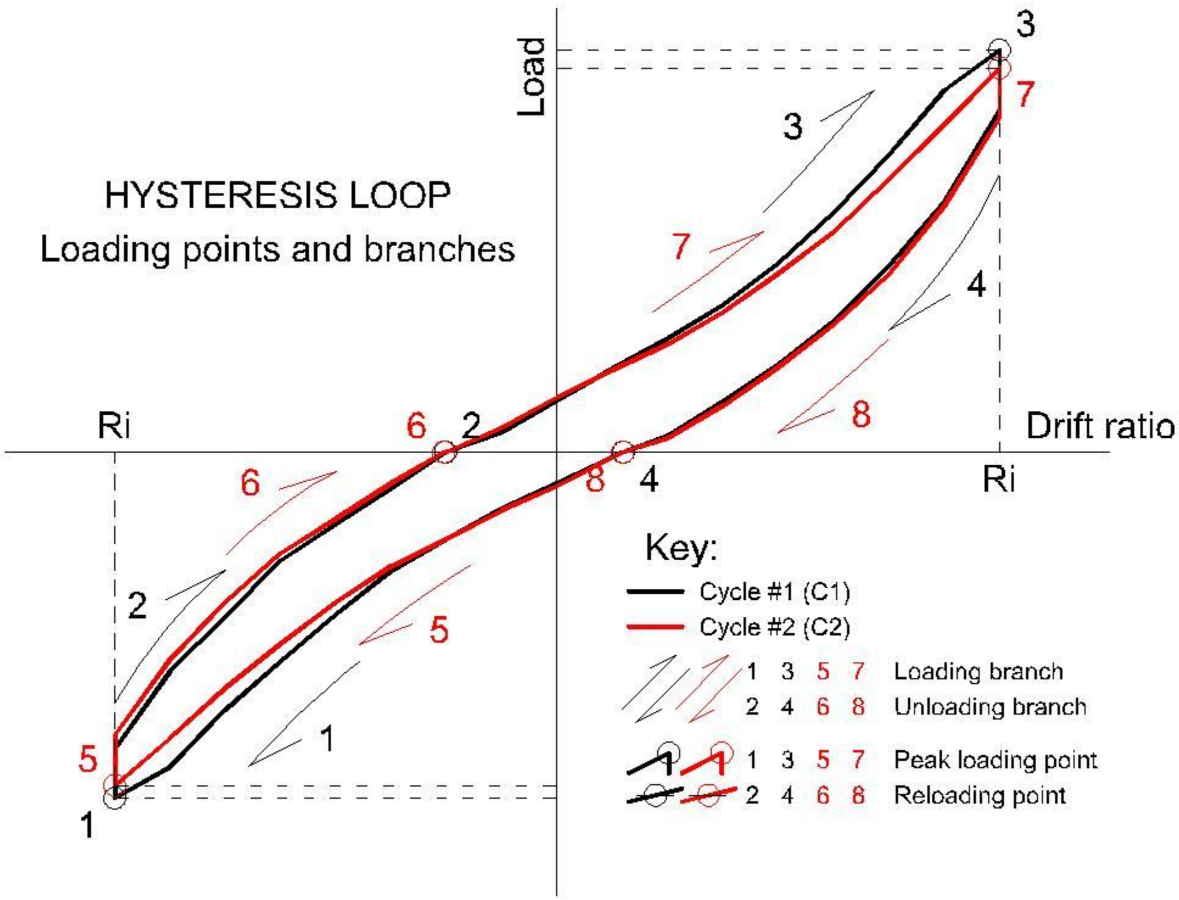








Loading points

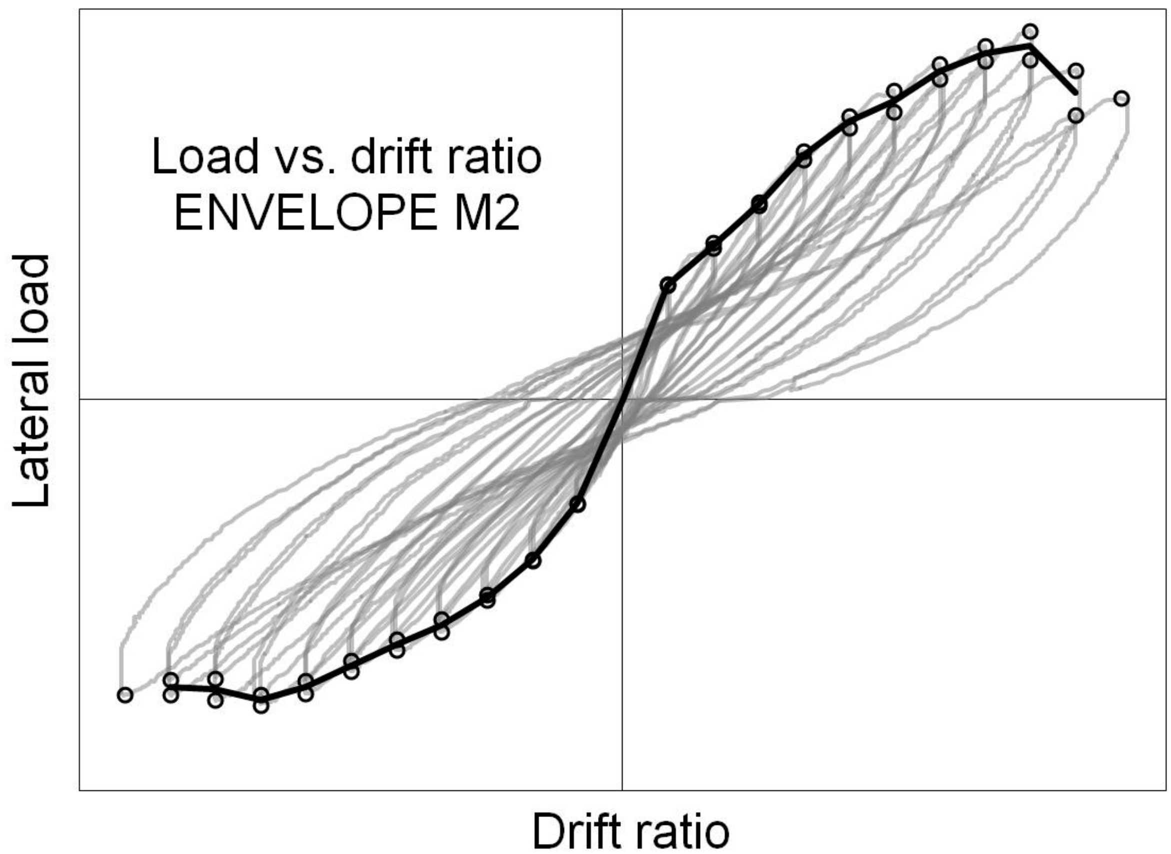
Cyclic envelope

Equiv. monotonic

Average loading

Backbone

Elasto-plastic



Loading points

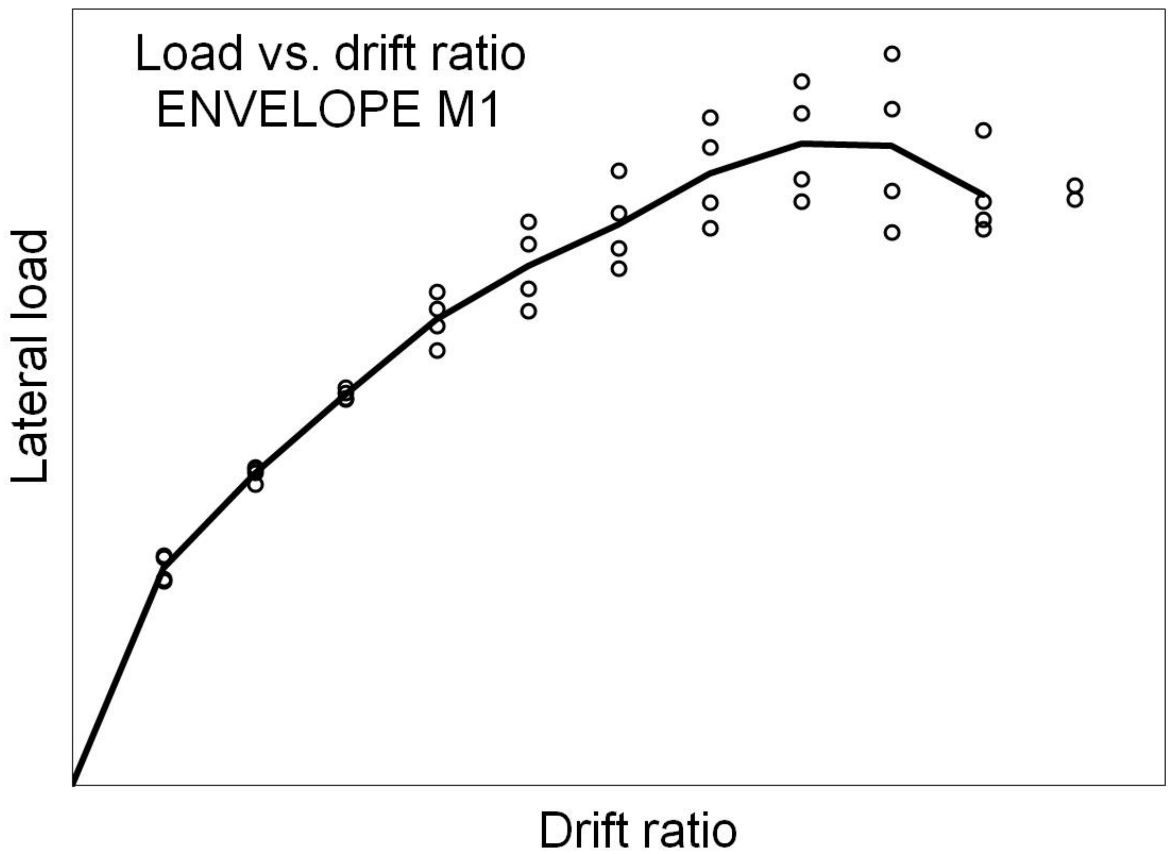
Cyclic envelope

Equiv. monotonic

Average loading

Backbone

Elasto-plastic



Loading points

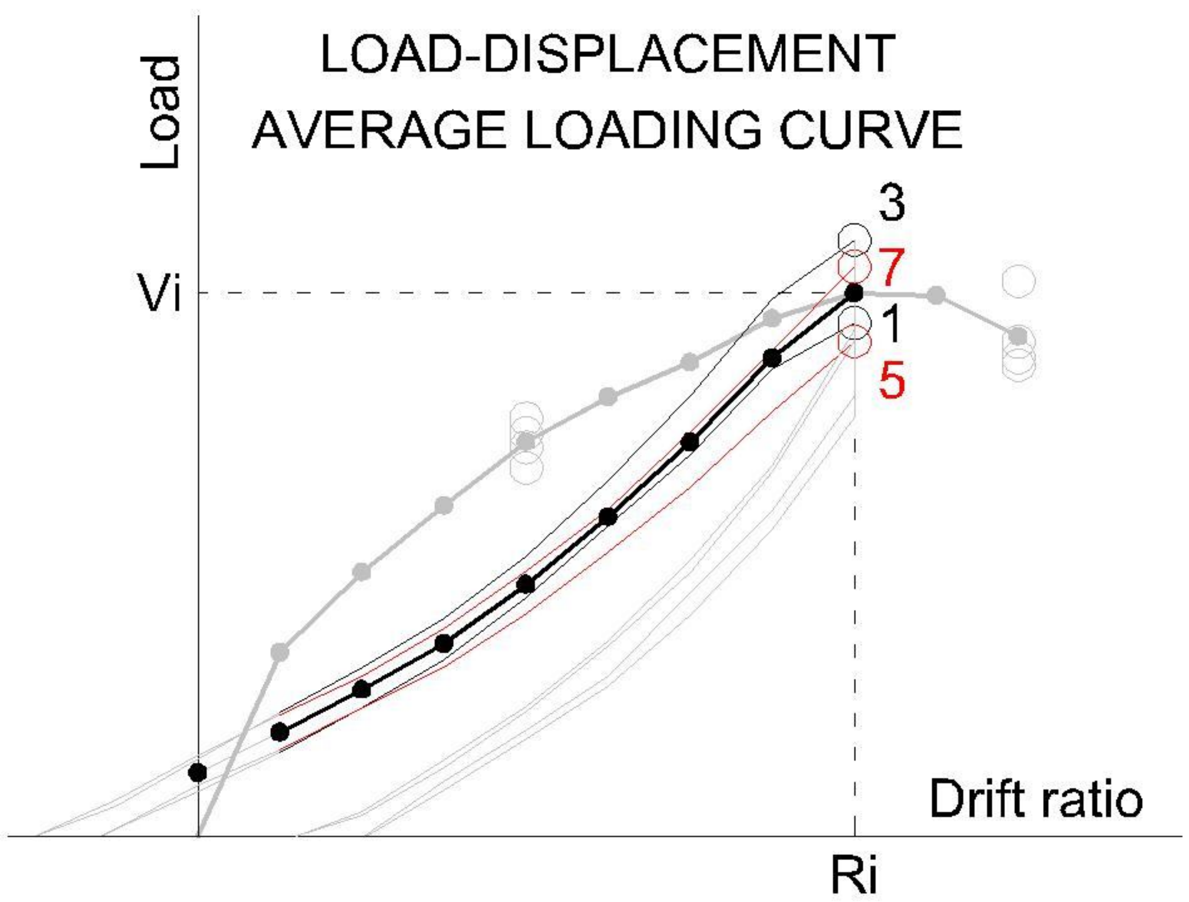
Cyclic envelope

Equiv. monotonic

Average loading

Backbone

Elasto-plastic



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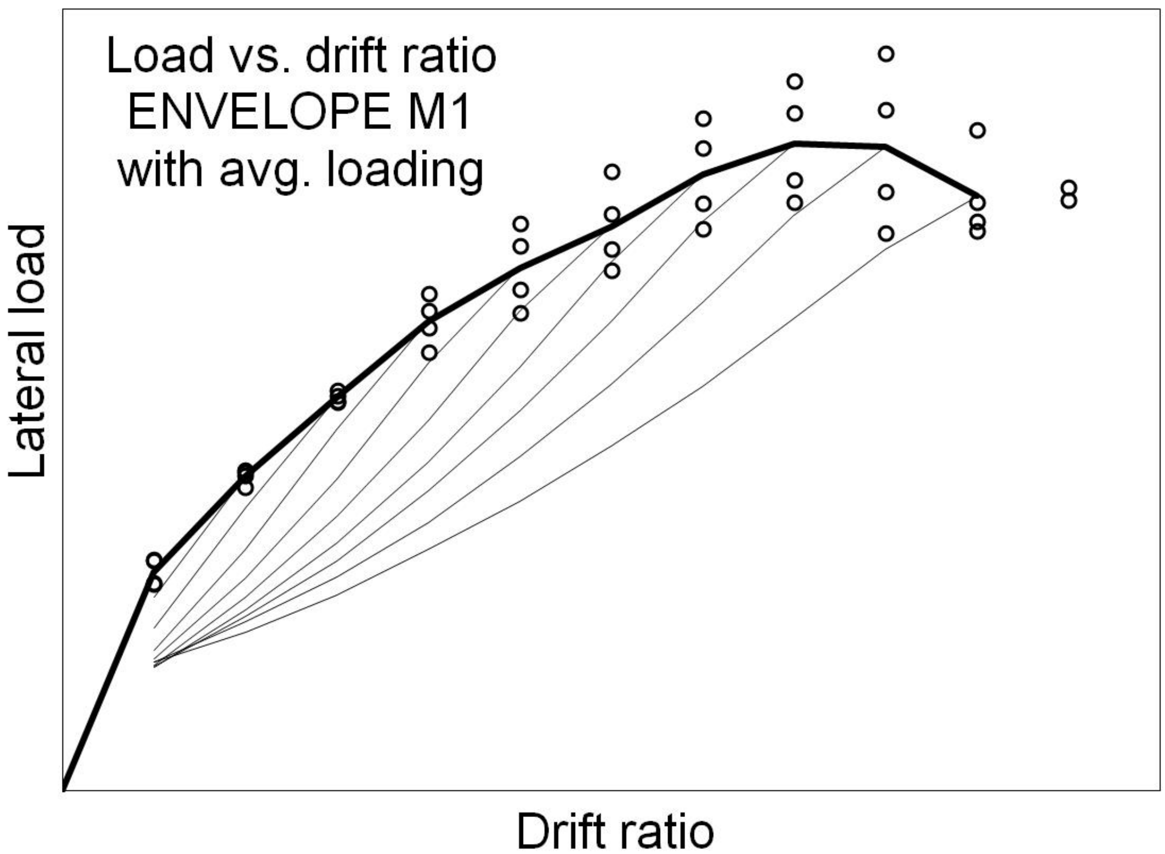


Loading points

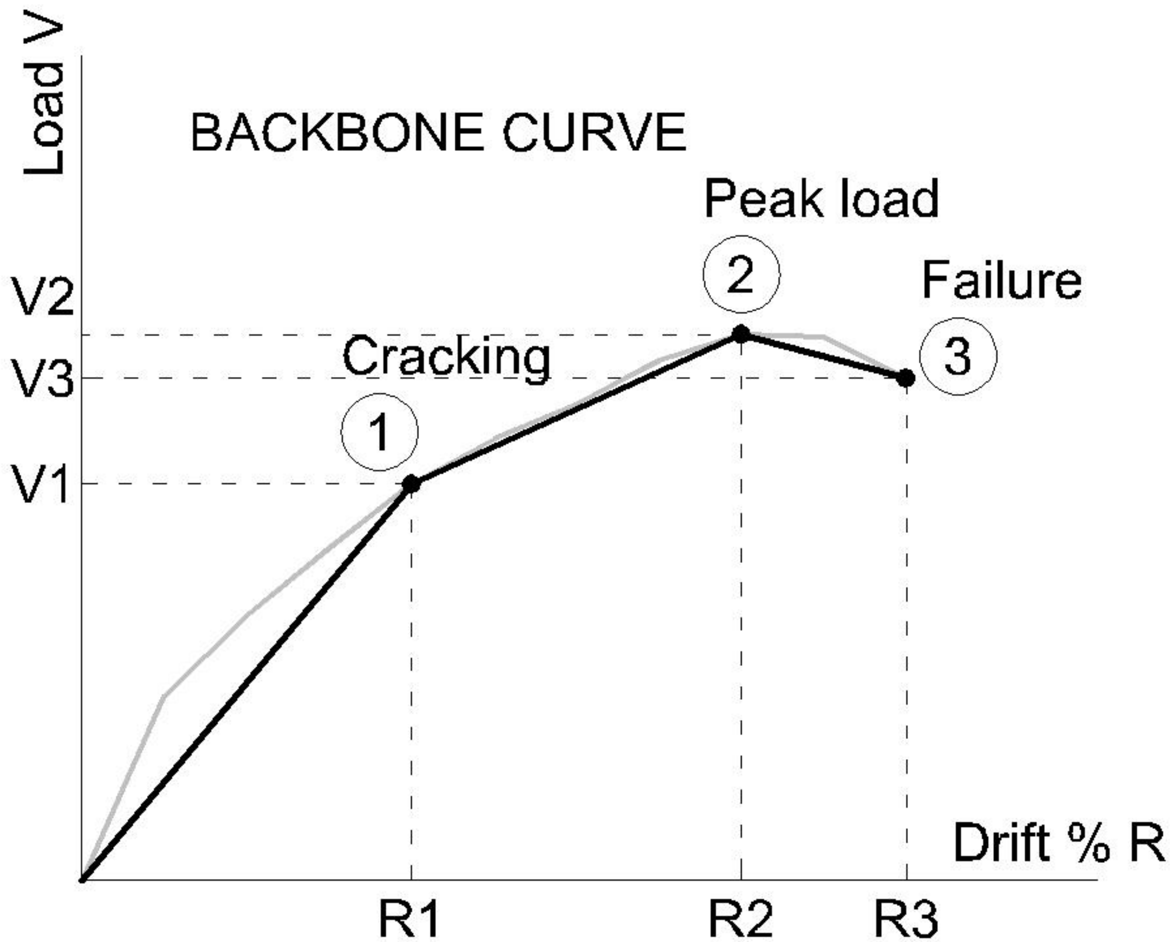
Cyclic envelope

Equiv. monotonic

Average loading

Backbone

Elasto-plastic



Loading points

Cyclic envelope

Equiv. monotonic

Average loading

Backbone

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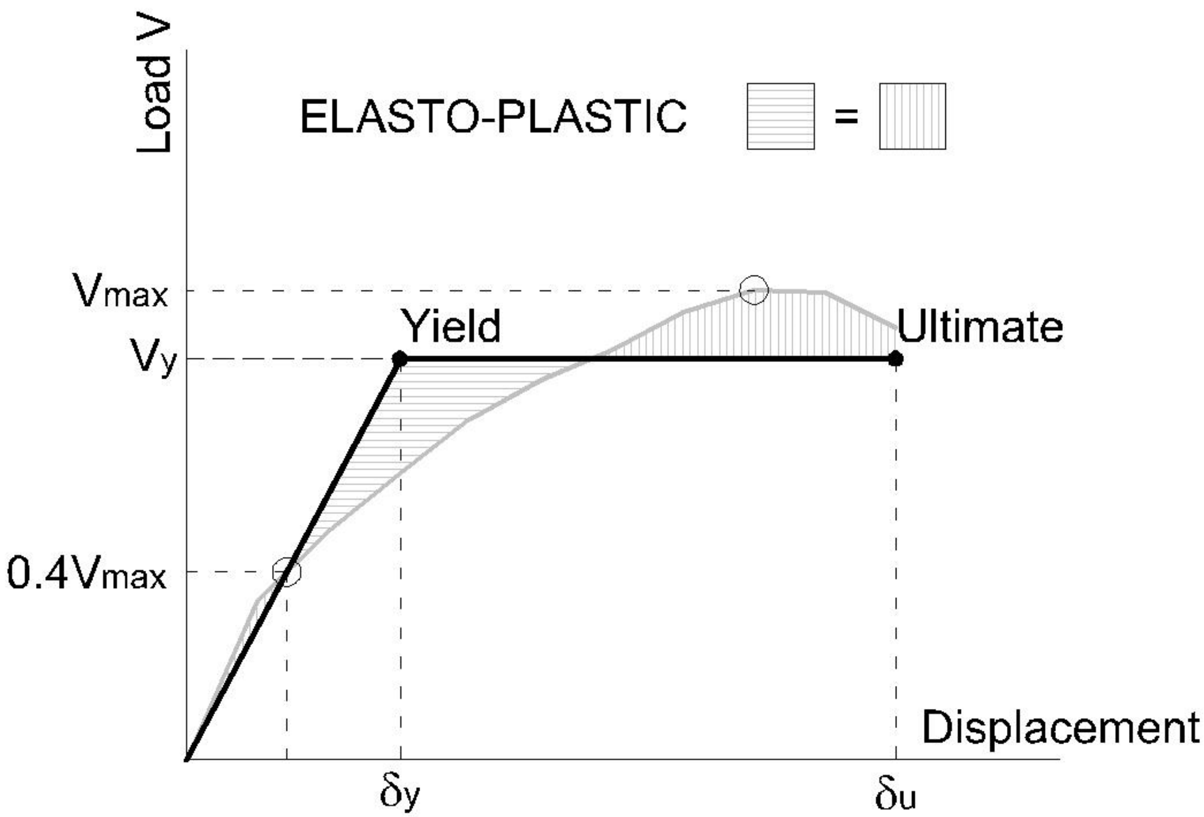


Loading points

Cyclic envelope

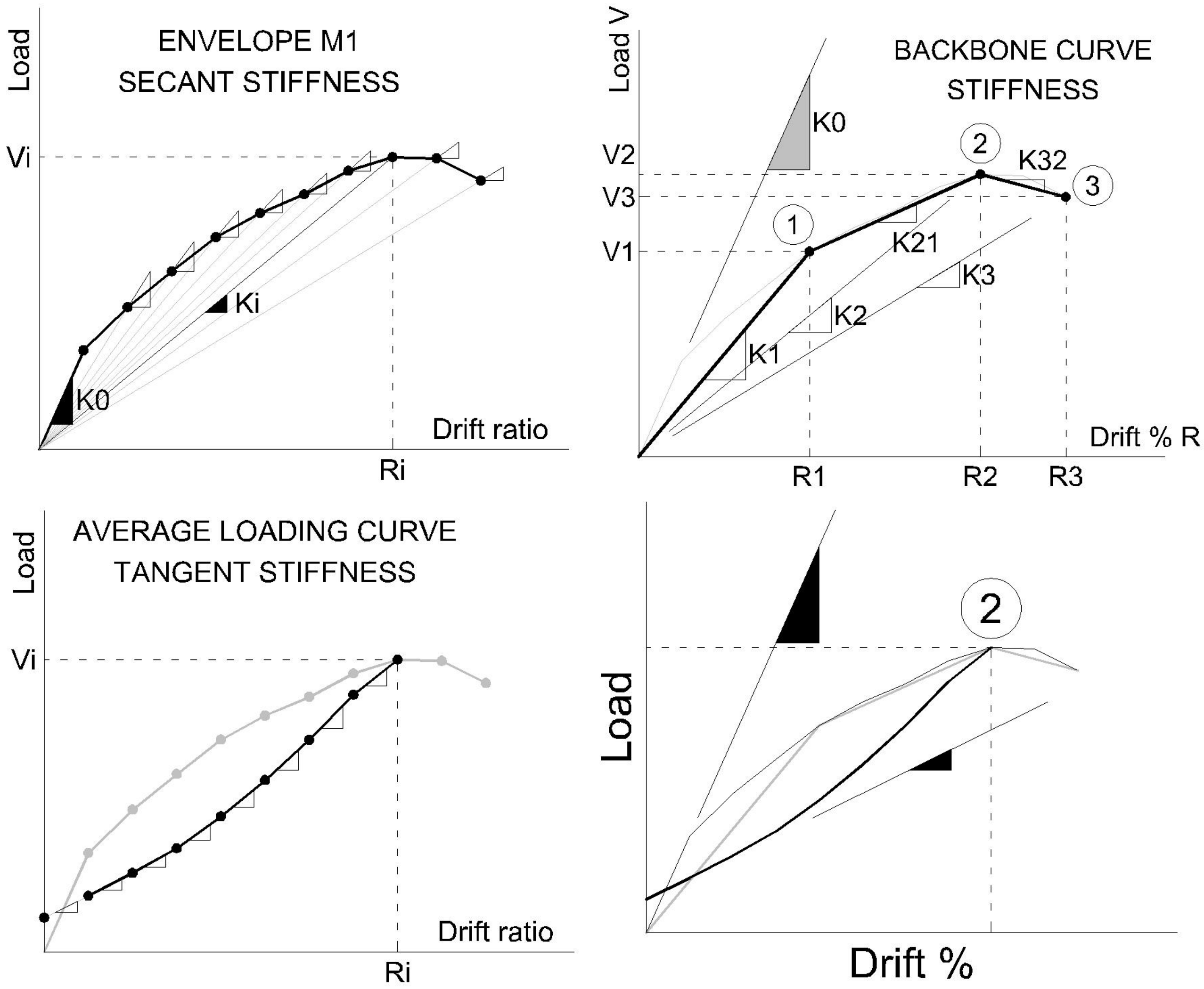
Equiv. monotonic

Average loading

Backbone

Elasto-plastic

PART A – STRUCTURAL REHABILITATION USING CFRP



Monotonic envelope stiffness

secant; tangent

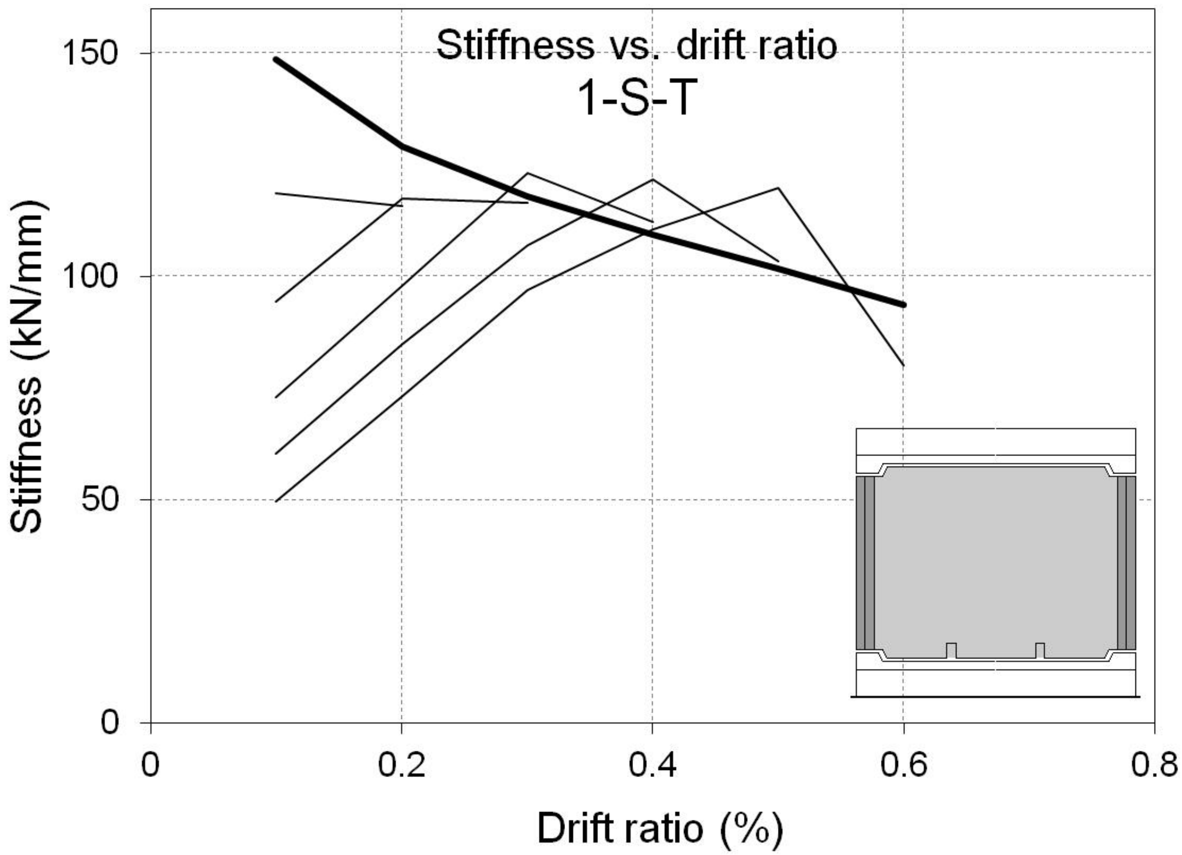
Backbone stiffness

secant; tangent

Average loading curve stiffness

secant; **tangent**

Reloading stiffness ratio

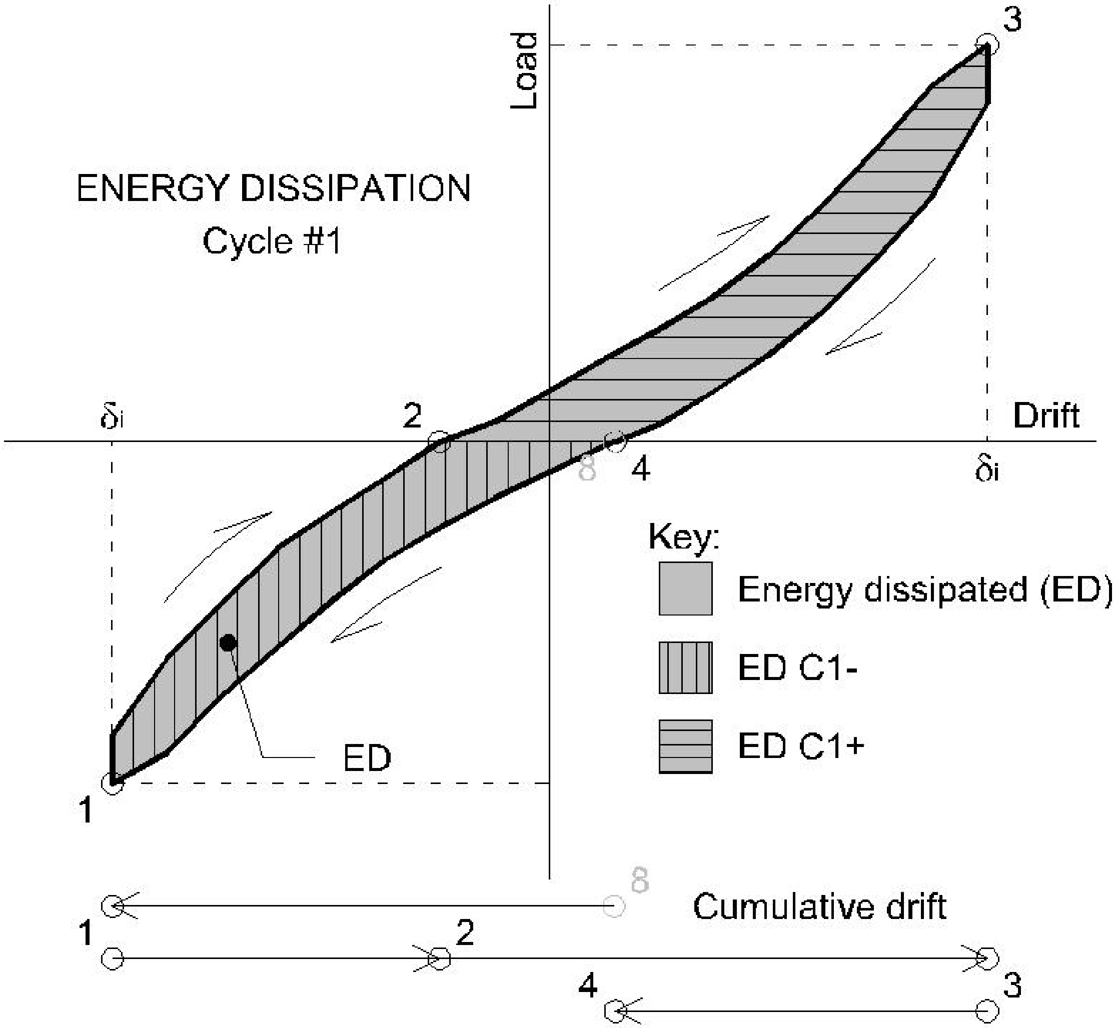


Secant stiffness

Tangent stiffness

Reloading stiffness ratio

Cyclic

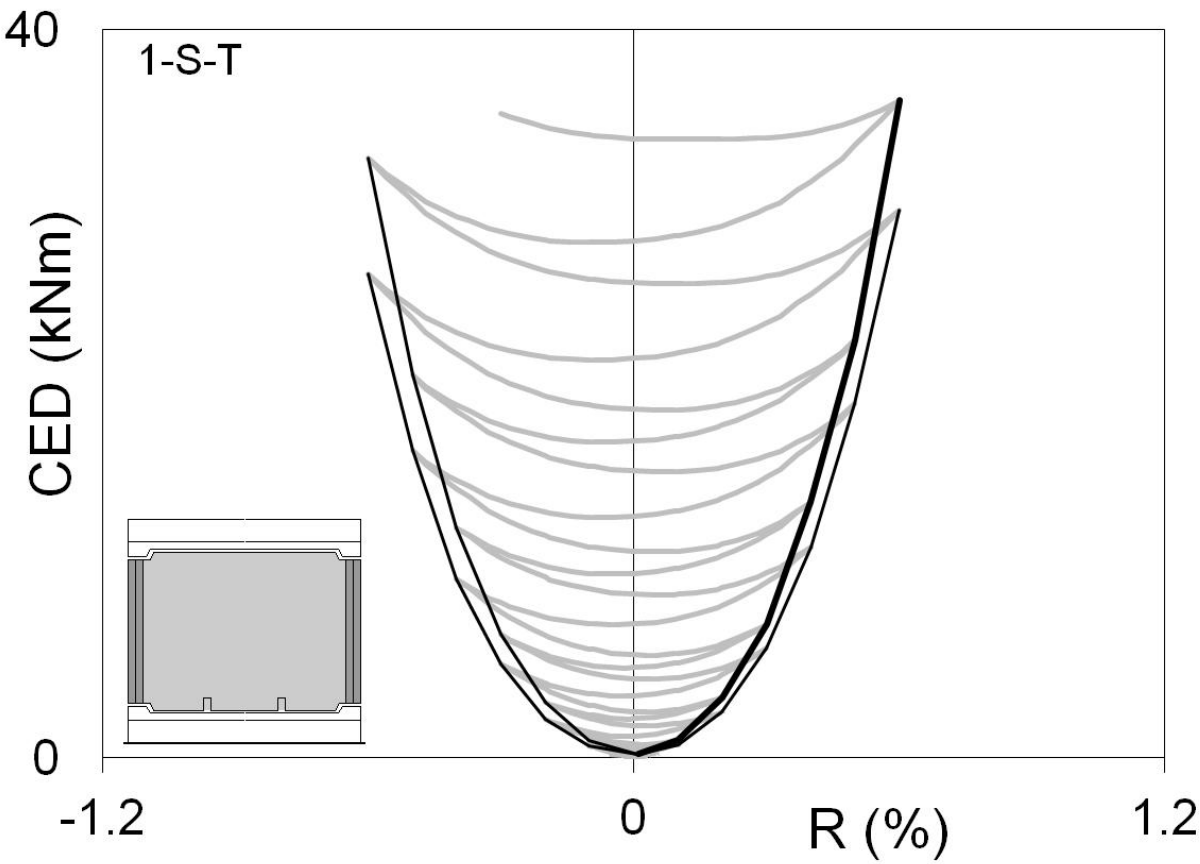


Cumulative dissipation 1

Cumulative dissipation 2

Dissipation rate CED/CD

Dissipation ratio



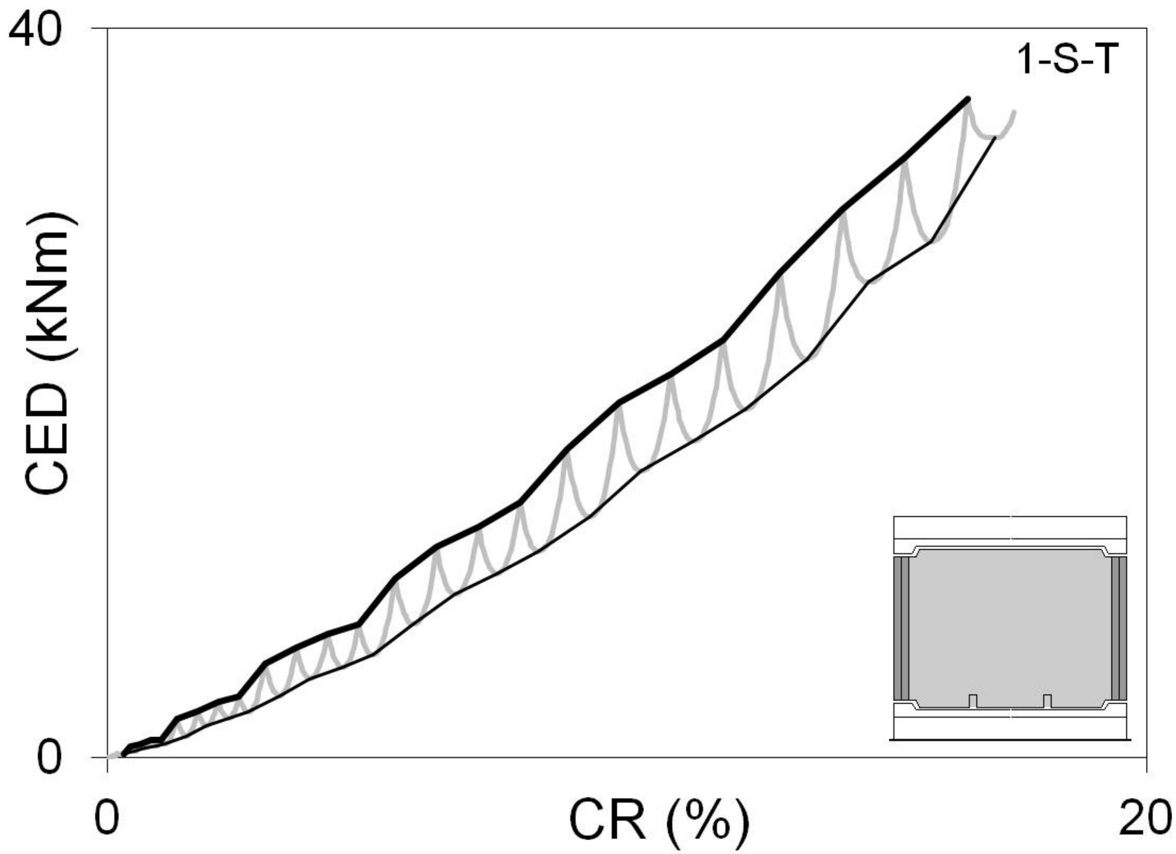
Cyclic

Cumulative dissipation 1

Cumulative dissipation 2

Dissipation rate CED/CD

Dissipation ratio



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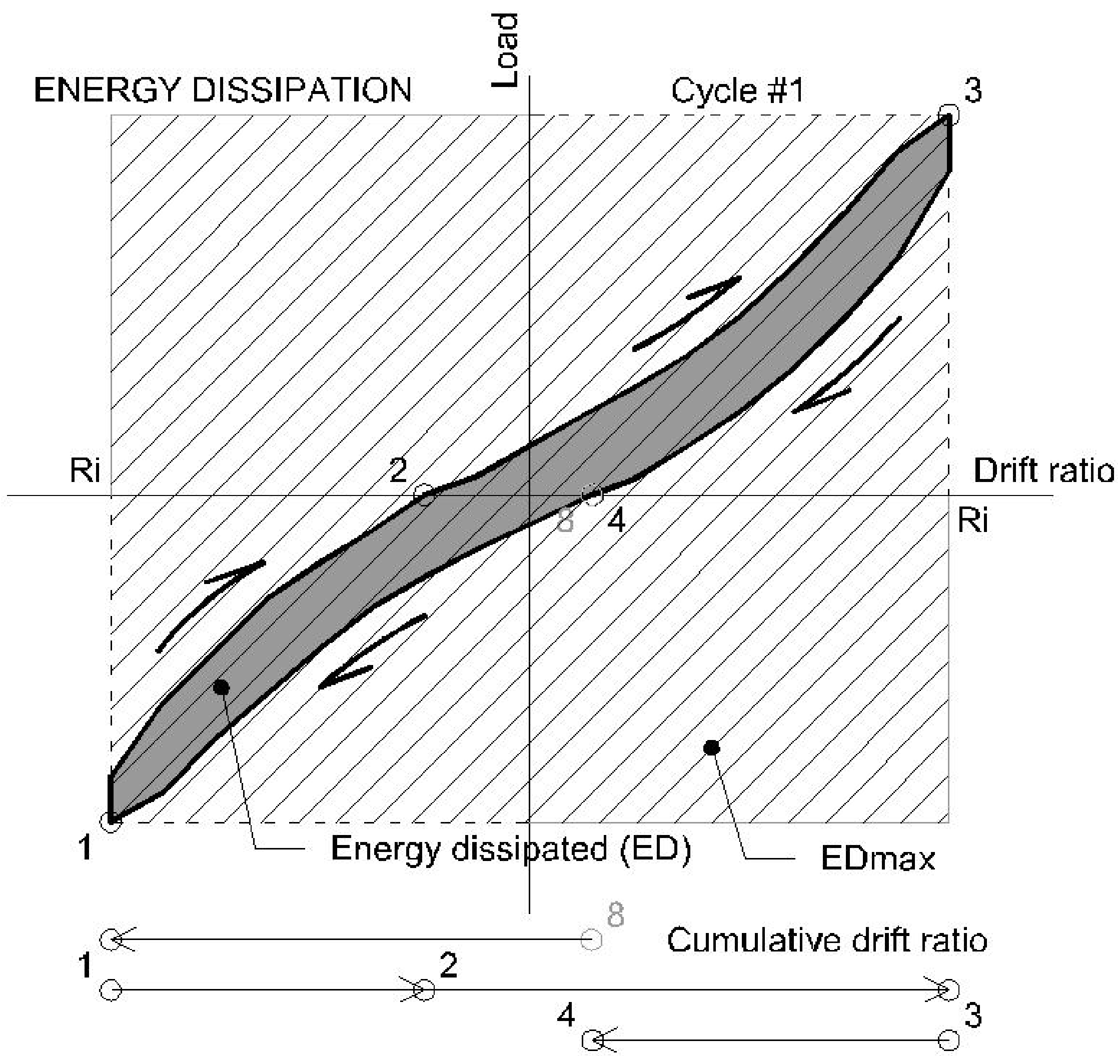

Cyclic

Cumulative dissipation 1

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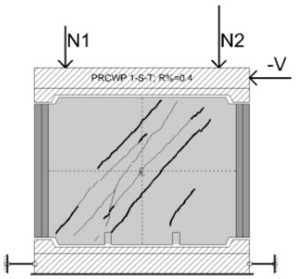
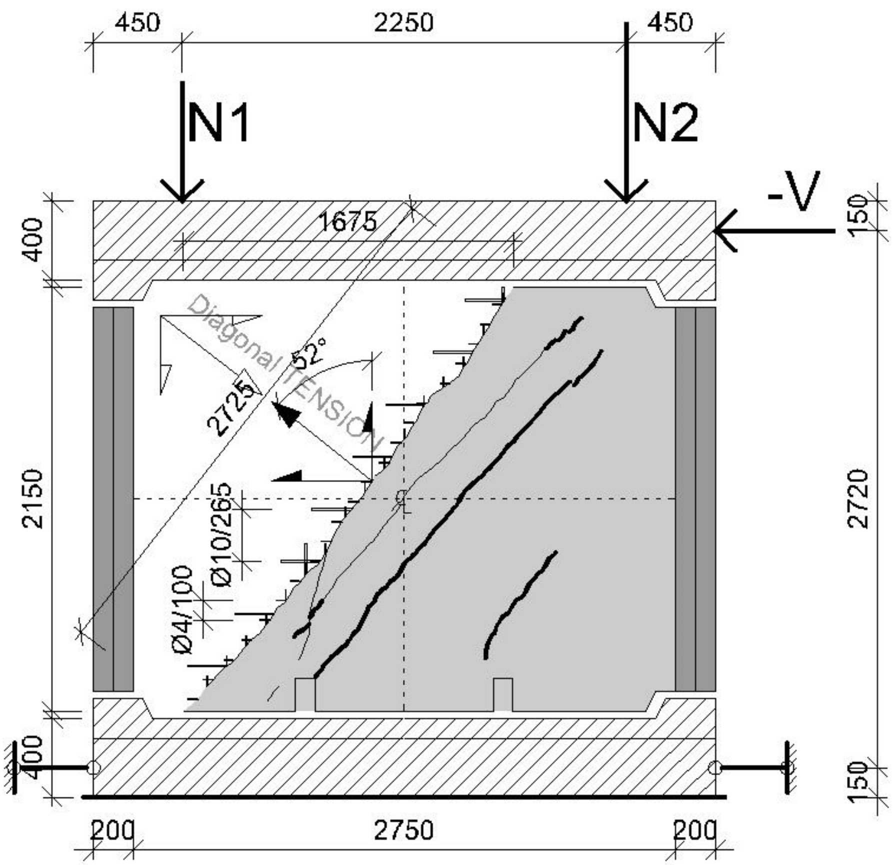
Dissipation ratio

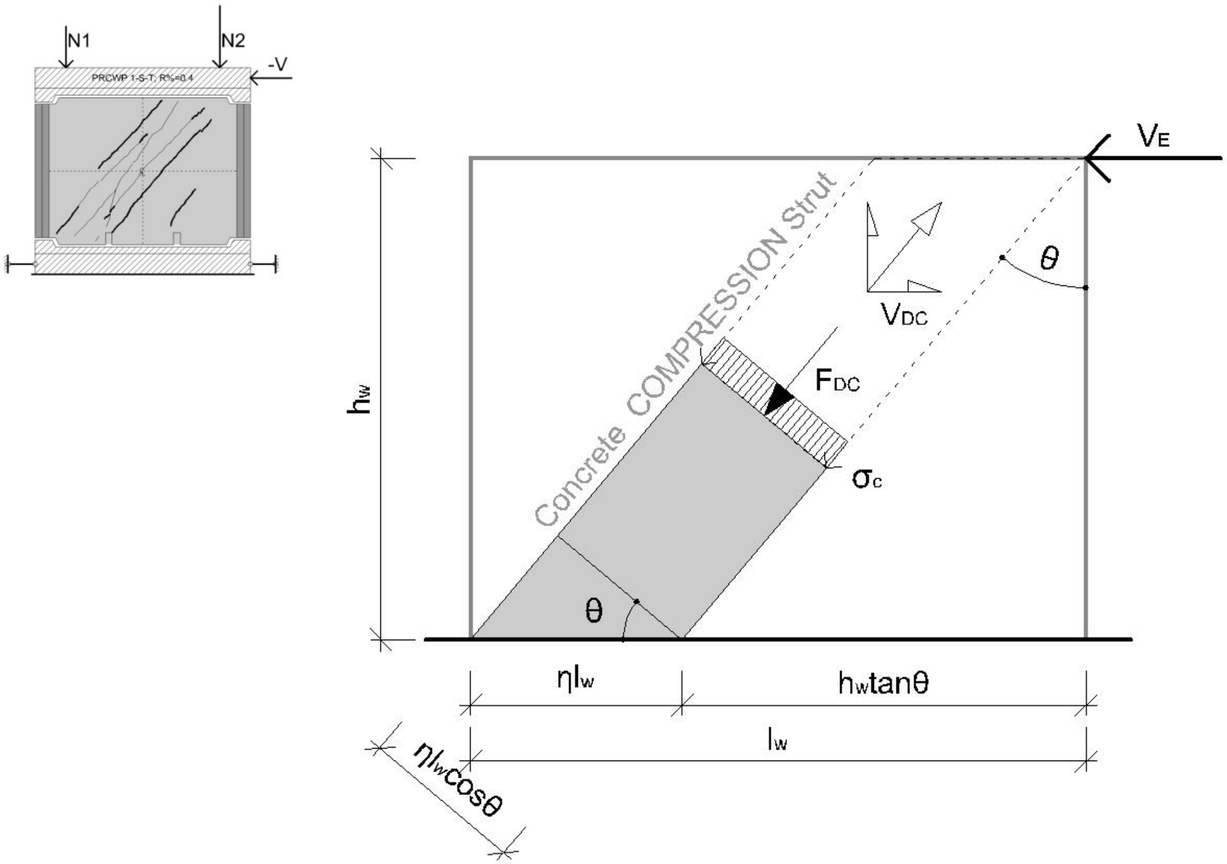




Diagonal tension
$V_{DT}=nA_s\sigma_s$
P/M ratio: 0.37

Diagonal compression
$V_{DC}=F_{DC}\sin\theta$
P/M ratio 0.82

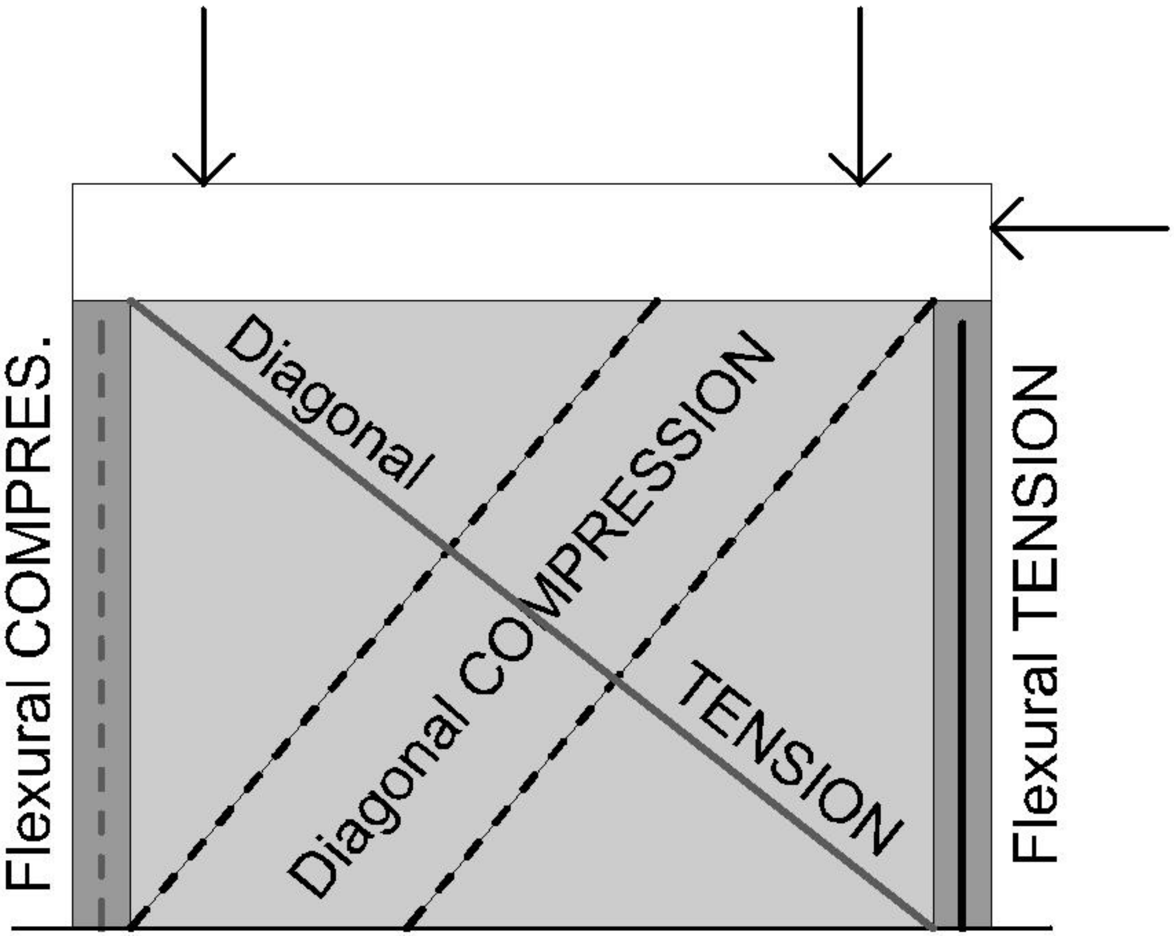
Shear transfer is compression dominated



Diagonal tension
$V_{DT}=nA_s\sigma_s$
P/M ratio 0.37

Diagonal compression
$V_{DC}=F_{DC}\sin\theta$
P/M ratio: 0.82

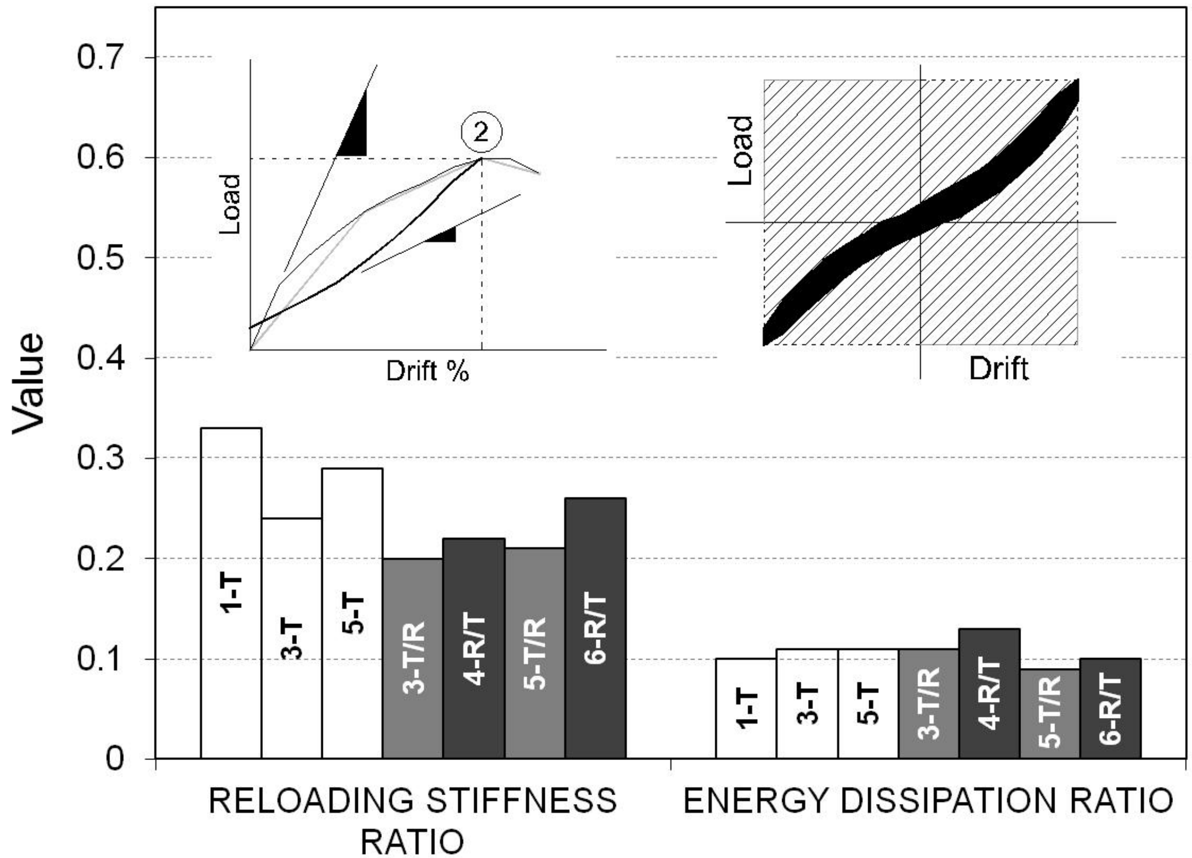
Shear transfer is compression dominated



Diagonal tension
$V_{DT}=nA_s\sigma_s$
P/M ratio 0.37

Diagonal compression
$V_{DC}=F_{DC}\sin\theta$
P/M ratio 0.82

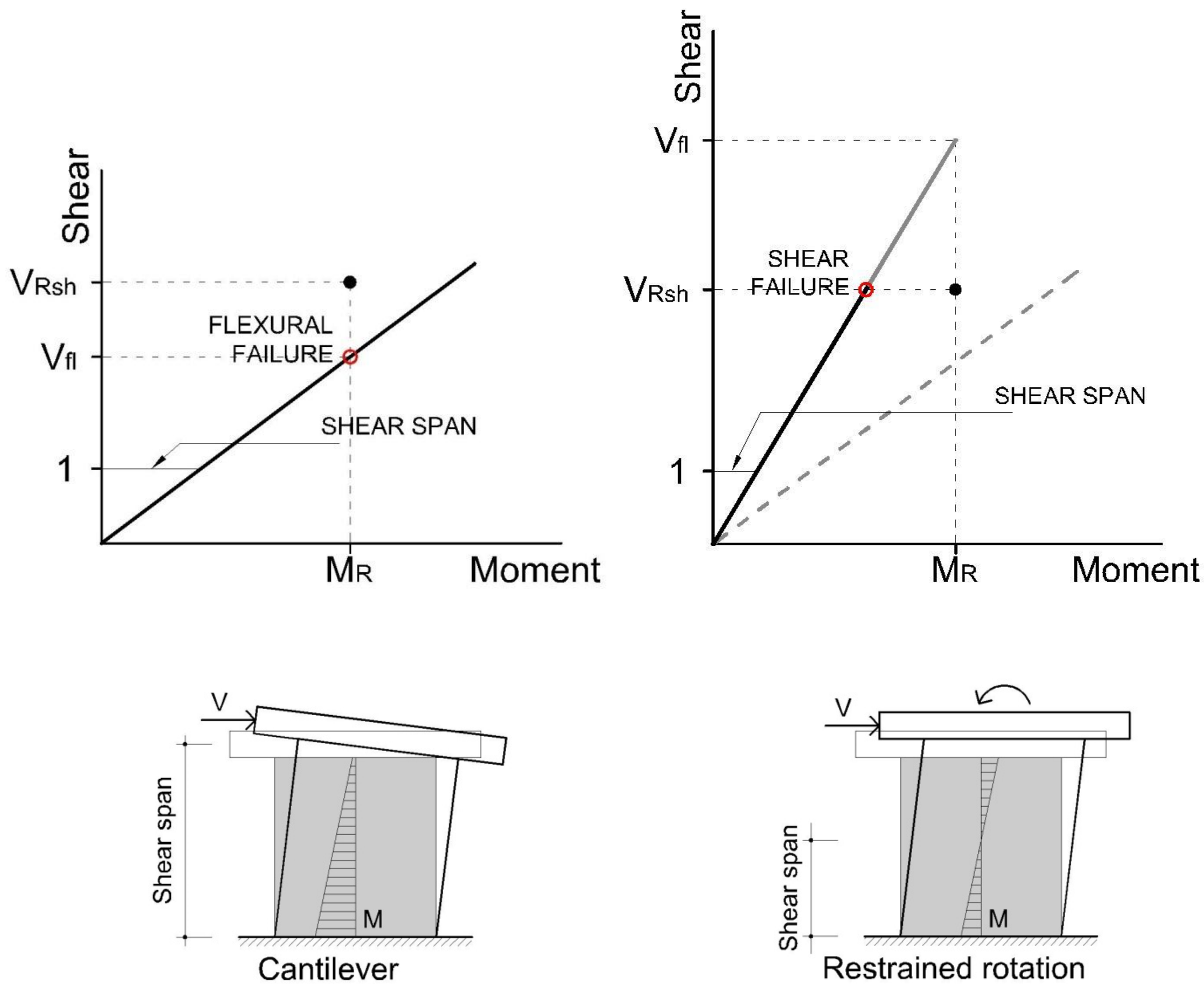
Shear transfer is compression dominated



Diagonal compression dominated shear response

Reloading stiffness ratio: 0.2÷0.33

Energy dissipation ratio: 10%



Seismic analysis

Laboratory investigations on cantilever walls tend to overestimate the shear span conditions relative to the as-built situation.

A reduced shear span condition may change the failure mode from flexural to shear for the same specimen.

Further investigations are necessary on this topic

Numerical analysis

A number of five RC shear walls were modelled and analysed in order to compare the behaviour, critical parts and failure mode with the tested ones.

The tested specimen had a lateral loading, reversed cyclic-displacement controlled, with increasing displacement amplitude load/displacement history. In comparison with the tested wall, in the modelled wall the lateral load were displacement controlled increments of 0.1 mm.

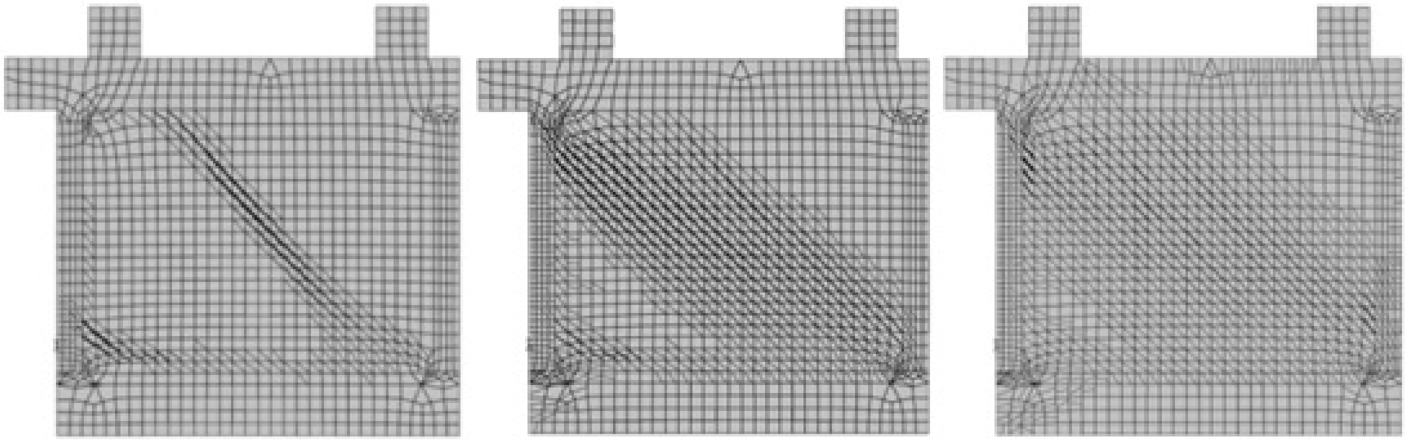
The axial loads for the tested specimen were composed of two parts, namely a constant and a variable part. For the modelled wall the axial loads consist of a constant part (95 kN + 95 kN) and an additional part composed of an axial load imposed on the left side by adding 10kN (for the large door opening) and 15 kN (for the narrow door opening) each step performed by the analysis in order to restrain the rocking rotation of the laterally loaded walls. The value of 10 kN and 15 kN were adopted considering the total load applied in the case of the tested specimen and the number of steps.

Bare solid wall results

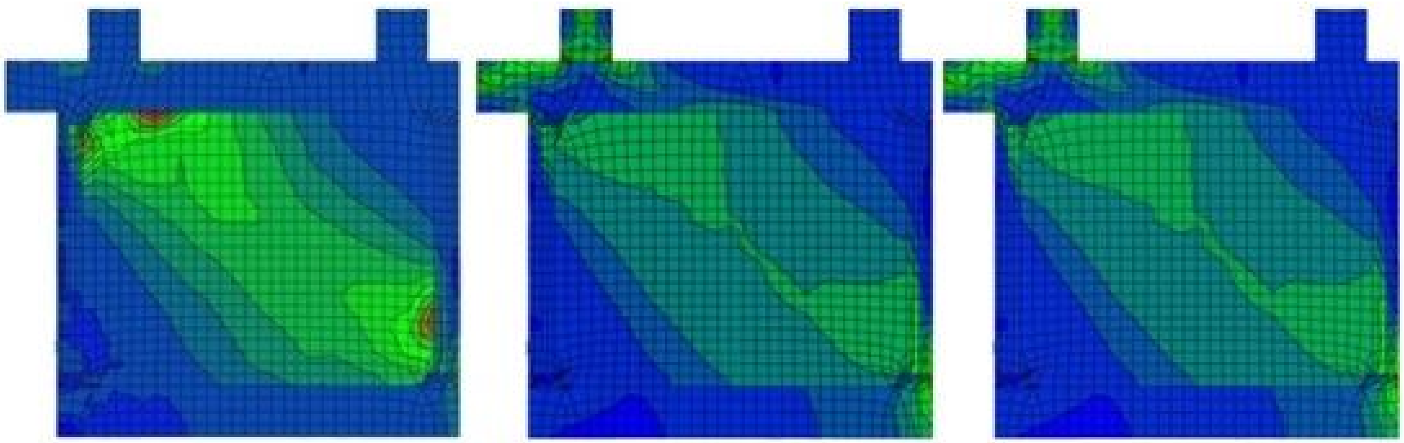
- ❑ **Evaluate the seismic performance**
- ❑ **Assess the weakening of the cut-outs**
- ❑ **Investigate the performance of the strengthening system**

[redacted]

Bare solid wall results



Crack state at (a) 0.70 mm, (b) 2.87 mm and (c) 7.43 mm drift level

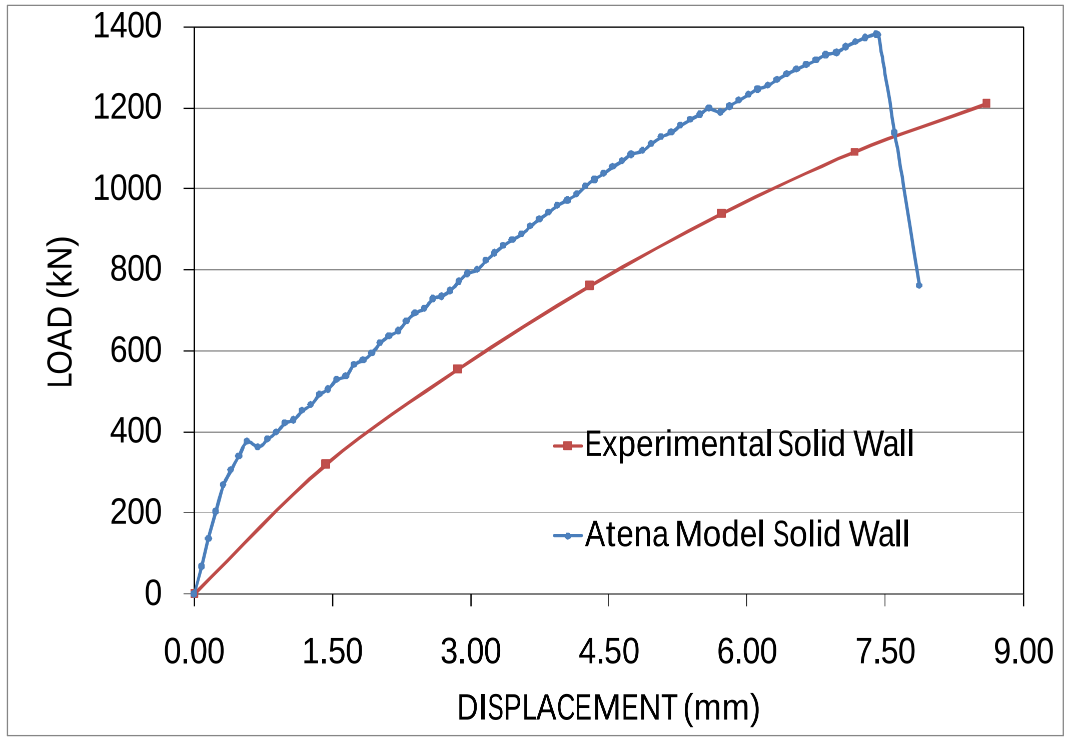


Compressive strain at (a) 7.4 mm, compressive stress at (b) 7.4 mm and (c) 7.43 mm

Bare solid wall results

- **Failure of the specimen takes place at 7.87 mm drift level by loosing 20% of its bearing capacity.**

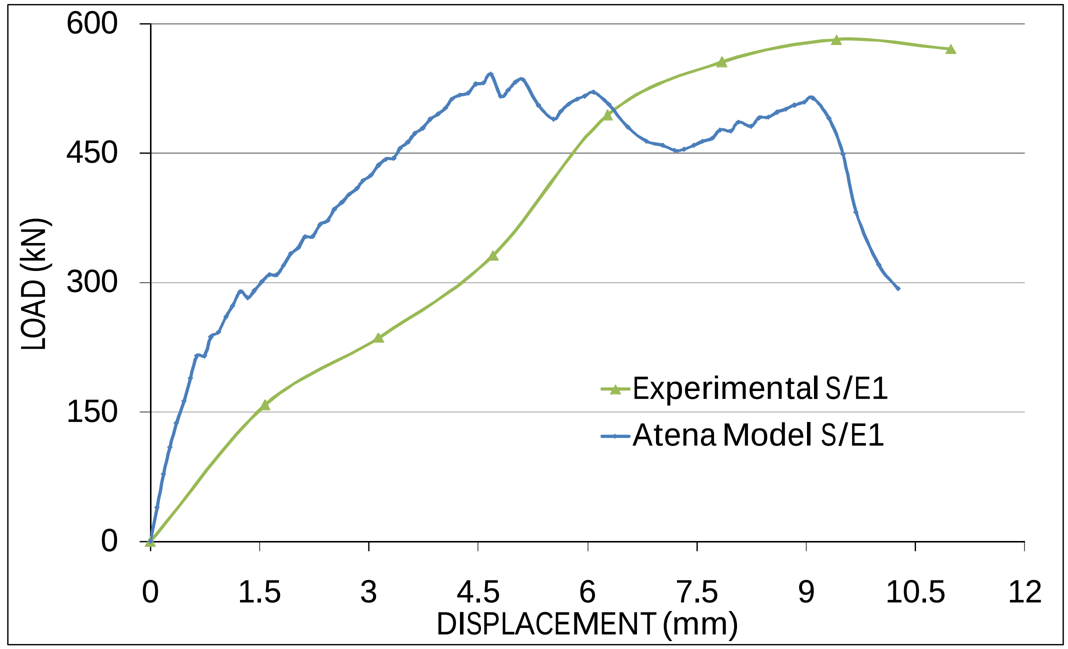
In terms of the maxi-mum load suppor-ted by the element, the mode-lled wall recorded 1380 kN while the tested specimen had 1210 kN. The maximum drift level obtained by the mode-lled solid wall was 7.87 mm while for the tested specimen was 8.6 mm.



Load displacement diagram

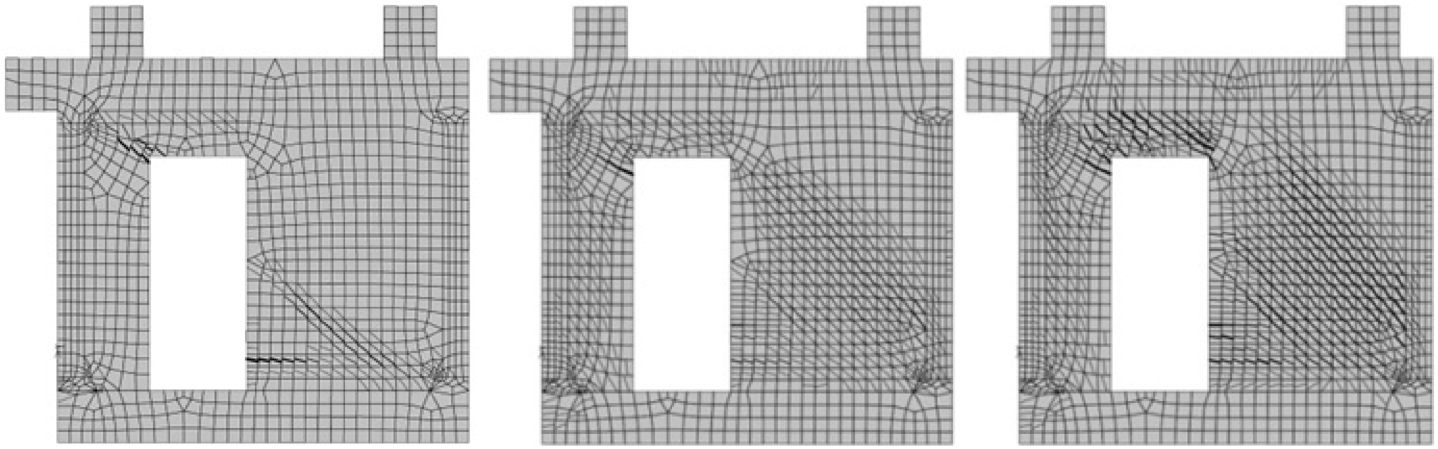
Wall with narrow door cut-out results

The load displacement diagram and the primary results of the tested and modelled wall with large door cut-out are presented in these two figures.

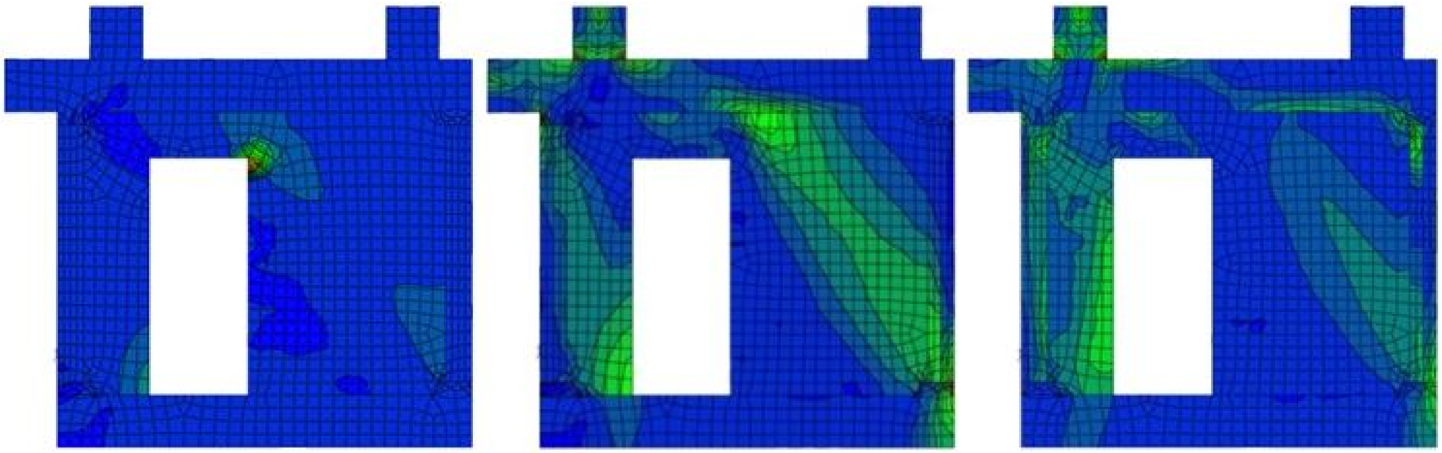


WALL WITH NARROW DOOR CUT-OUT RESULTS			
Element	First diagonal crack	Maximum lateral load imposed	Drift level corresponding to max lateral load imposed
Tested	δ = 6.28 ÷ 7.85 mm drift level	581.8 kN	9.42 mm
Modelled	δ = 1.23 mm drift level	541.8 kN	4.7 mm

Wall with narrow door cut-out results



Crack state at (a) - 1.23 mm, (b) - 4.67 mm and (c) - 6.29 mm drift level



Compressive strain at (a) - 5.02 mm, compressive stress at (b) - 5.54 and (c) - 9.68 mm

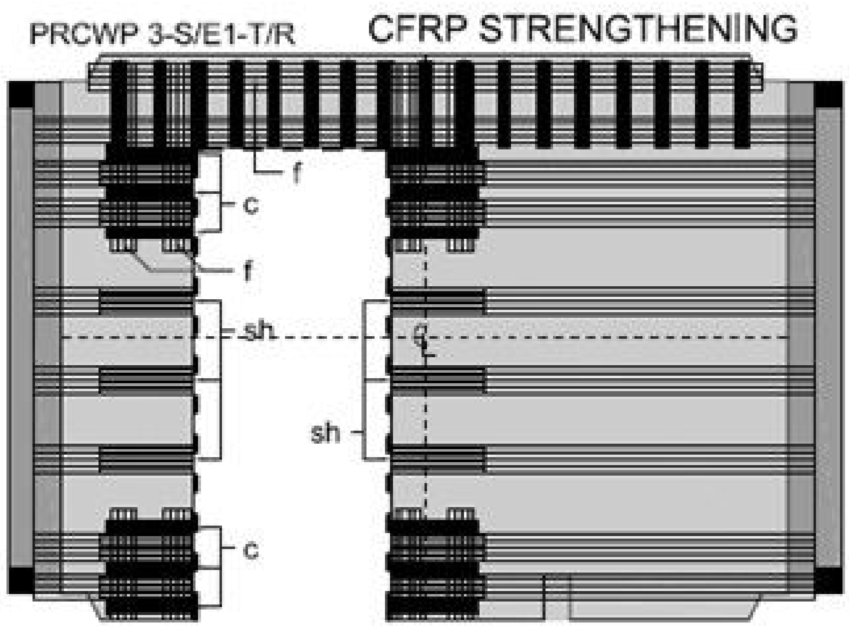
Post-damage strengthened wall with narrow door cut-out results

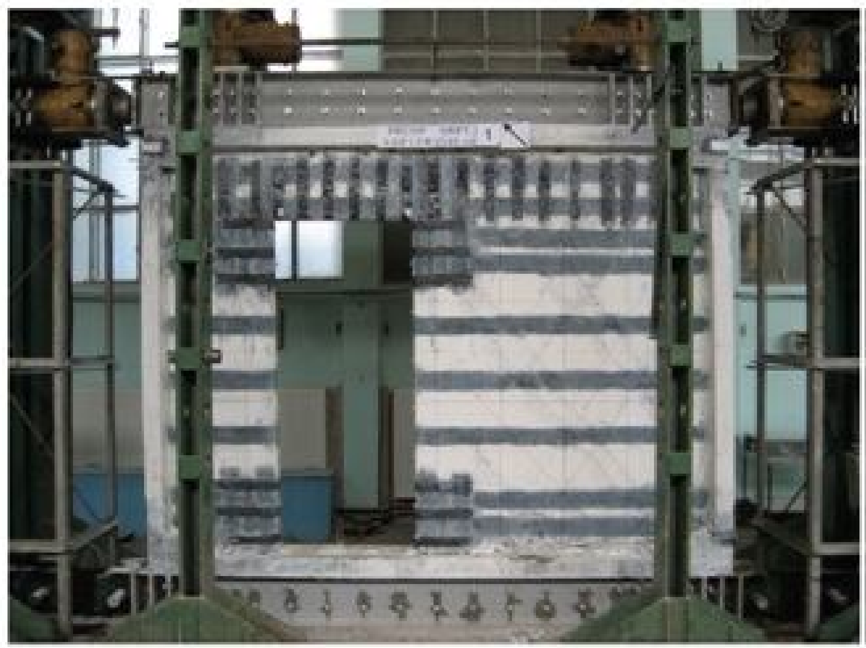
The overall objectives of the strengthening were to counterbalance the weakening incurred in a solid wall as a result of a door cut-out.

The strengthening strategy was divided into three directions:

- **to offer flexural capacity along the vertical and horizontal edges of the cut-out opening,**
- **to increase the shear capacity of the wall piers, and**
- **to provide confinement effect at the cut-out opening corners**

Post-damage strengthened wall with narrow door cut-out results

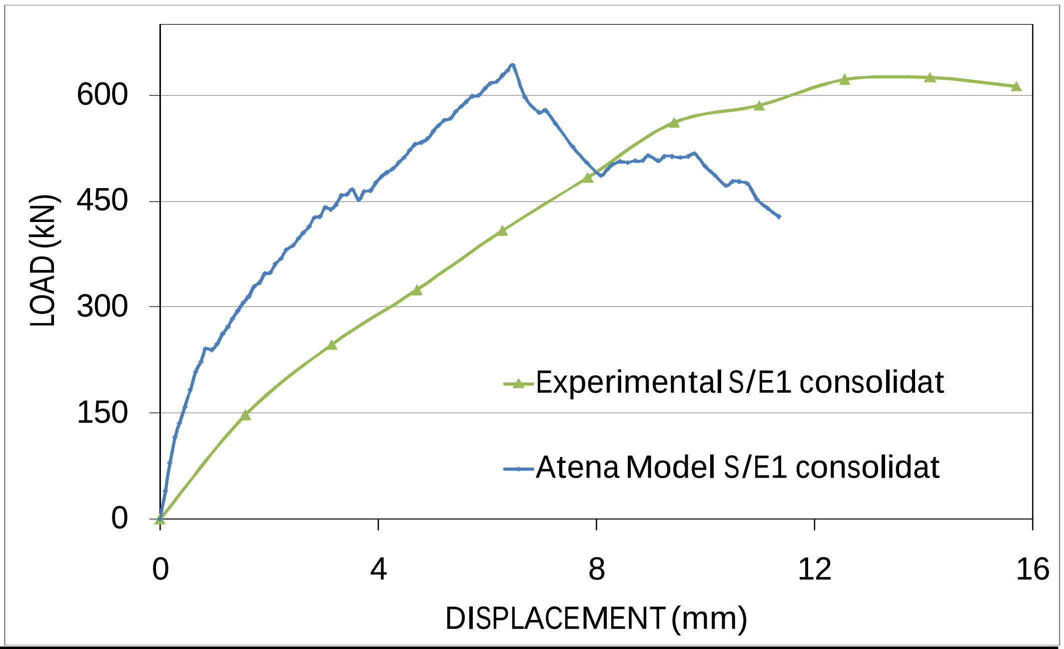




CFRP strengthening strategy for S/E1-T/R specimen

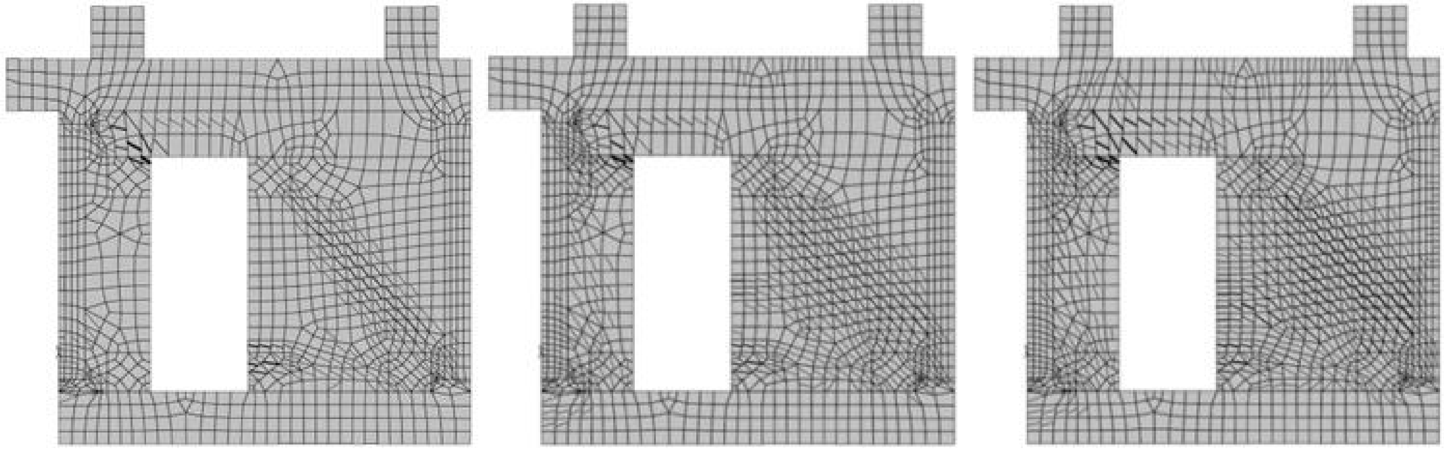
Post-damage strengthened wall with narrow door cut-out results

The load displacement diagram and the primary results of the tested and modelled wall with large door cut-out are presented in these two figures.

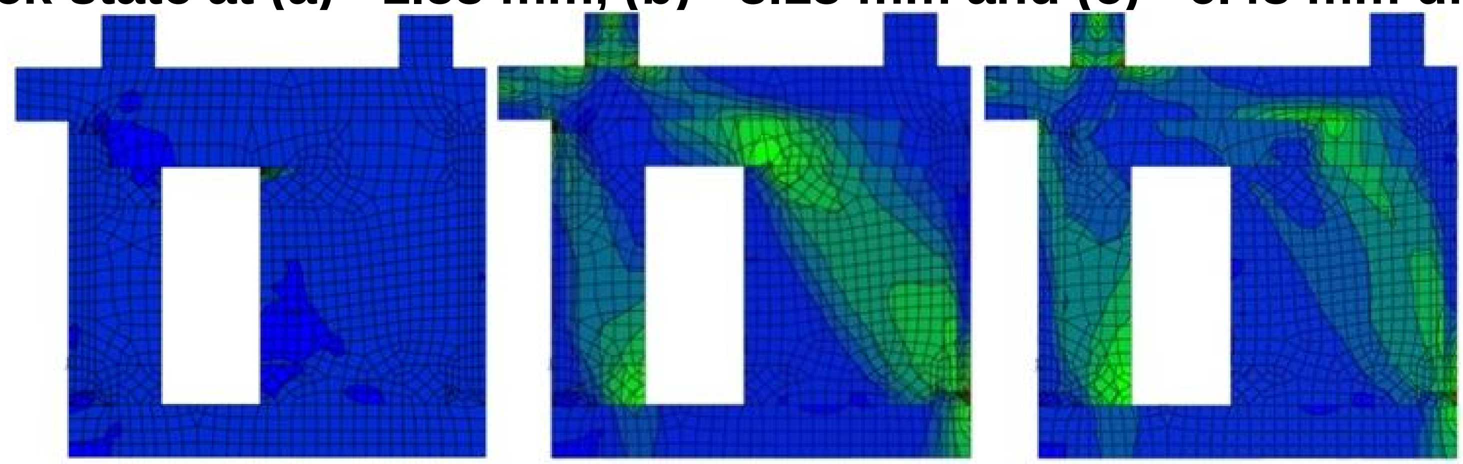


POST-DAMAGE STRENGTHENED WALL WITH NARROW DOOR CUT-OUT RESULTS				
Element	First diagonal crack	FRP failure initiation	Maximum lateral load imposed	Drift level corresponding to max lateral load imposed
Tested	δ = 3.14 mm drift level	6.28 mm	625.3 kN	14.13 mm
Modelled	δ = 1.53 mm drift level	6.48 mm	642 kN	6.48 mm

Post-damage strengthened wall with narrow door cut-out results



Crack state at (a) - 1.53 mm, (b) - 3.13 mm and (c) - 6.48 mm drift level

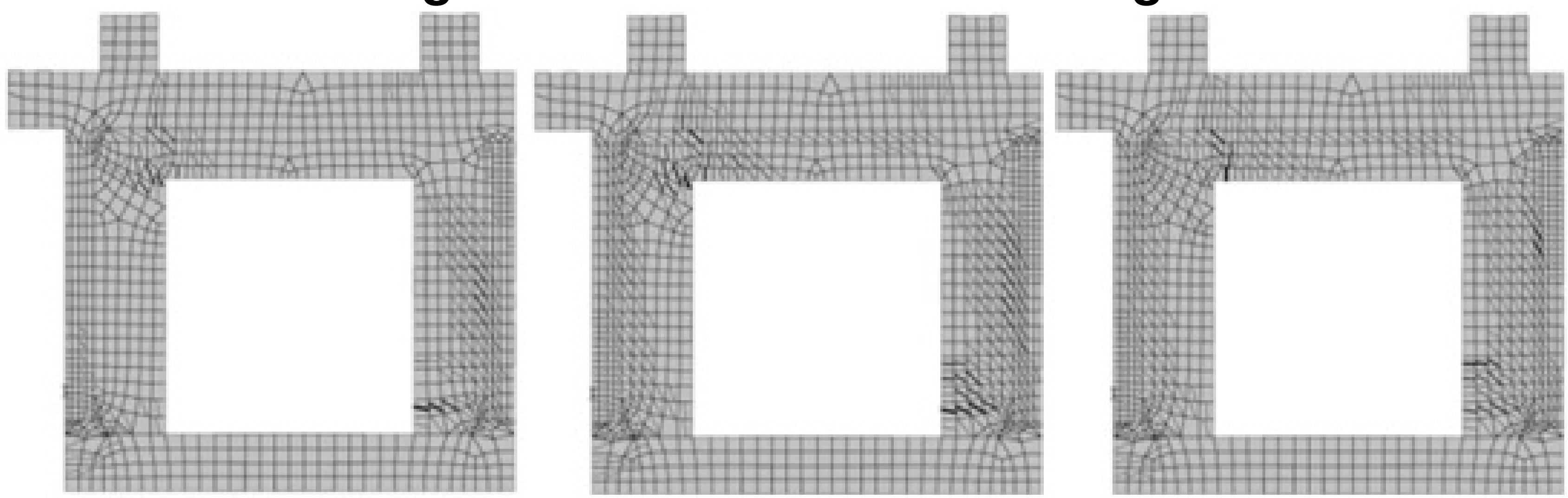


Compressive strain at (a) - 6.38 mm, compressive stress at (b) - 6.38 and (c) - 7.8 mm

Wall with large door cut-out results

- **The figure below shows the crack state of the wall with large door**

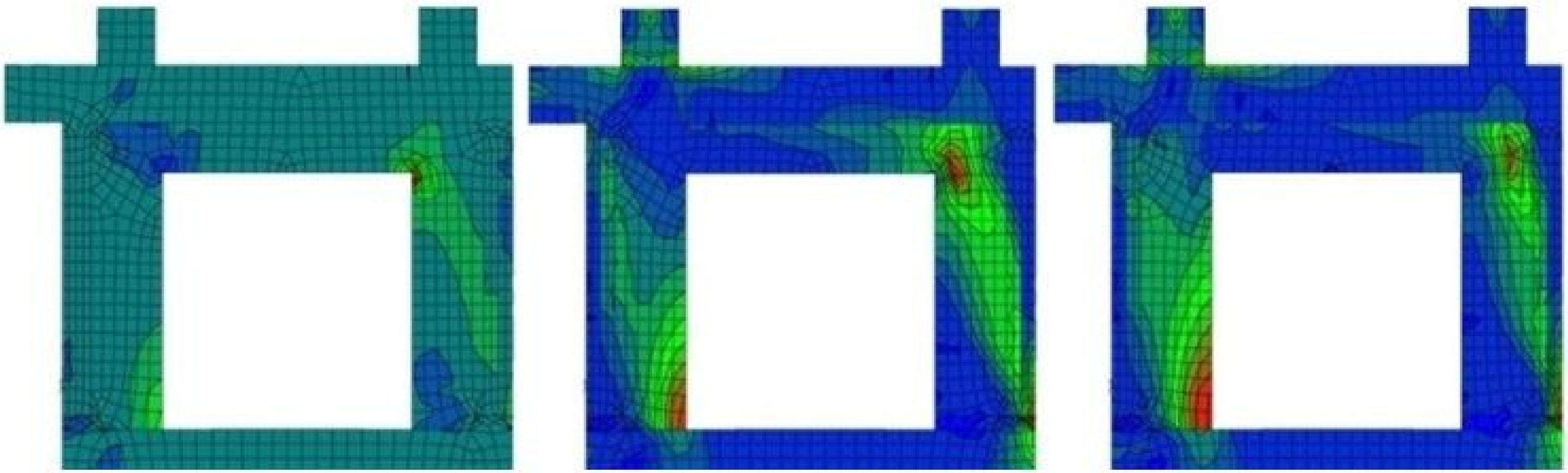
cut-out according to the numerical modelling.



(a) (b) (c)

Crack state at (a) - 3.3 mm, (b) - 5.85 mm and (c) - 7.49 mm drift level

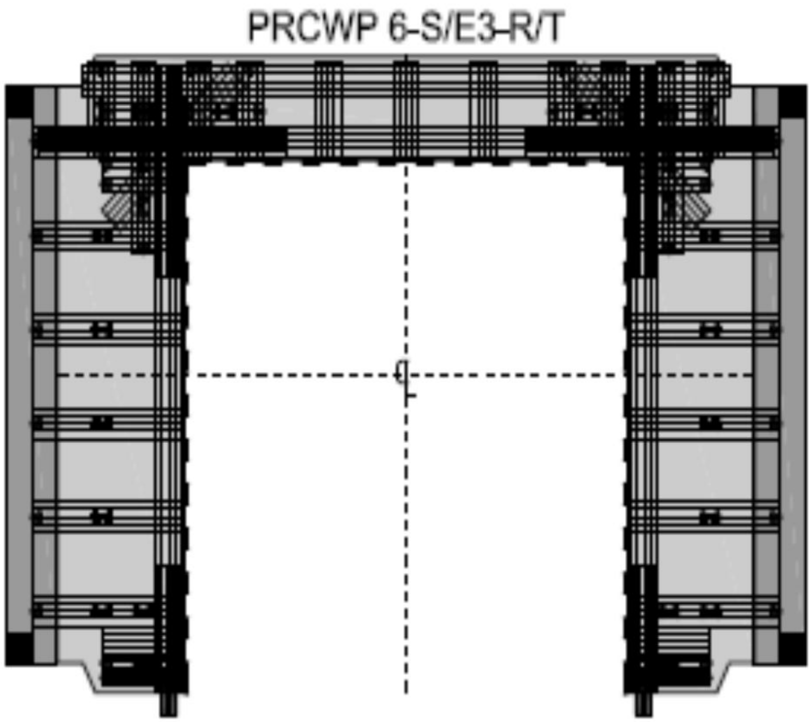
Wall with large door cut-out results

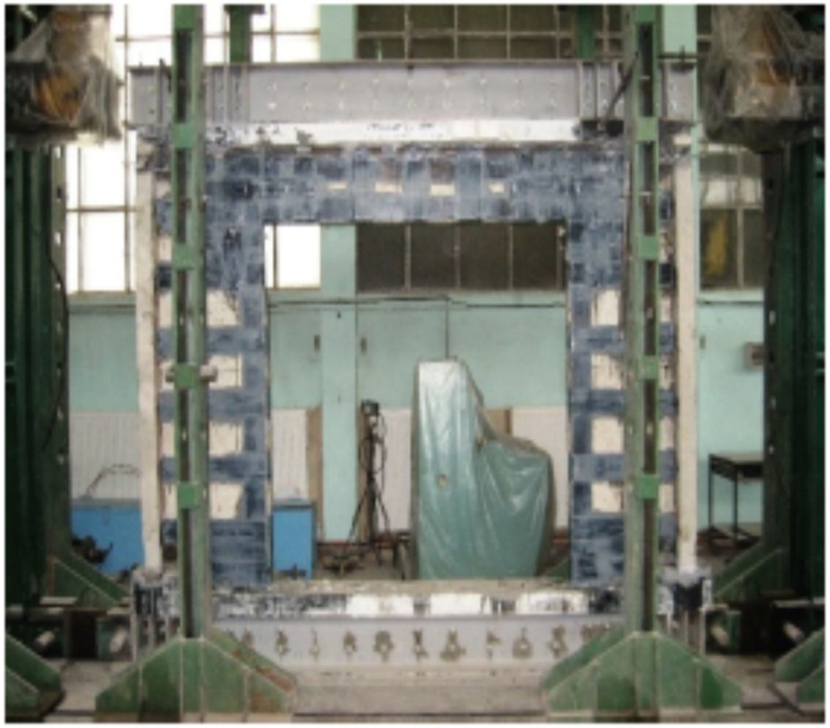


(a) (b) (c)

Compressive strain at (a) 5.25 mm, compressive stress at (b) 5.85 mm and (c) 7.49 mm

Prior-to-damage strengthened wall with large door cut-out

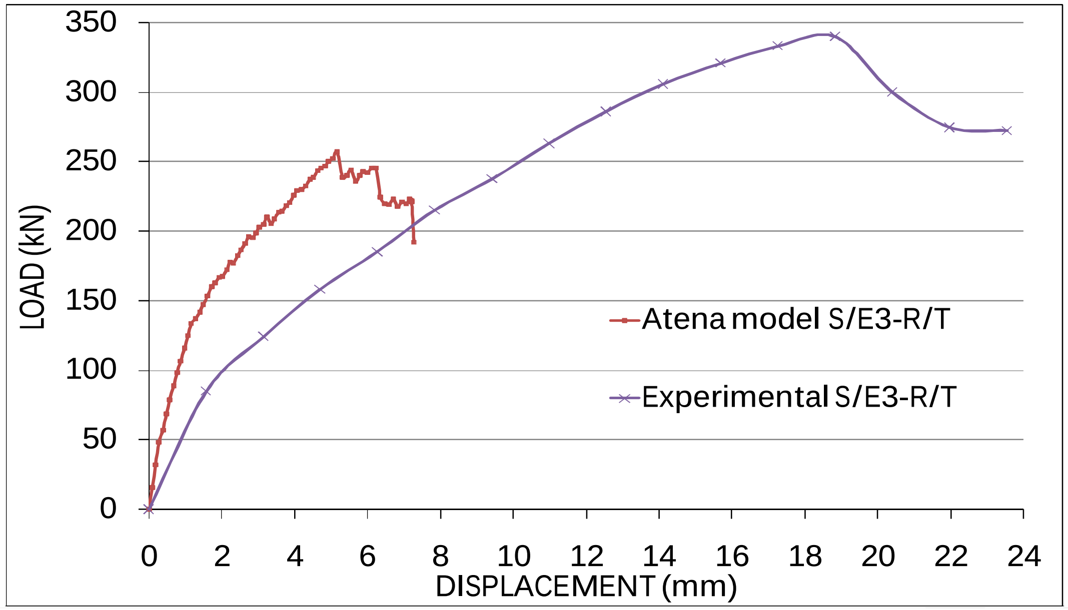




CFRP strengthening strategy for S/E3-R/T specimen [1]

Prior-to-damage strengthened wall with large door cut out

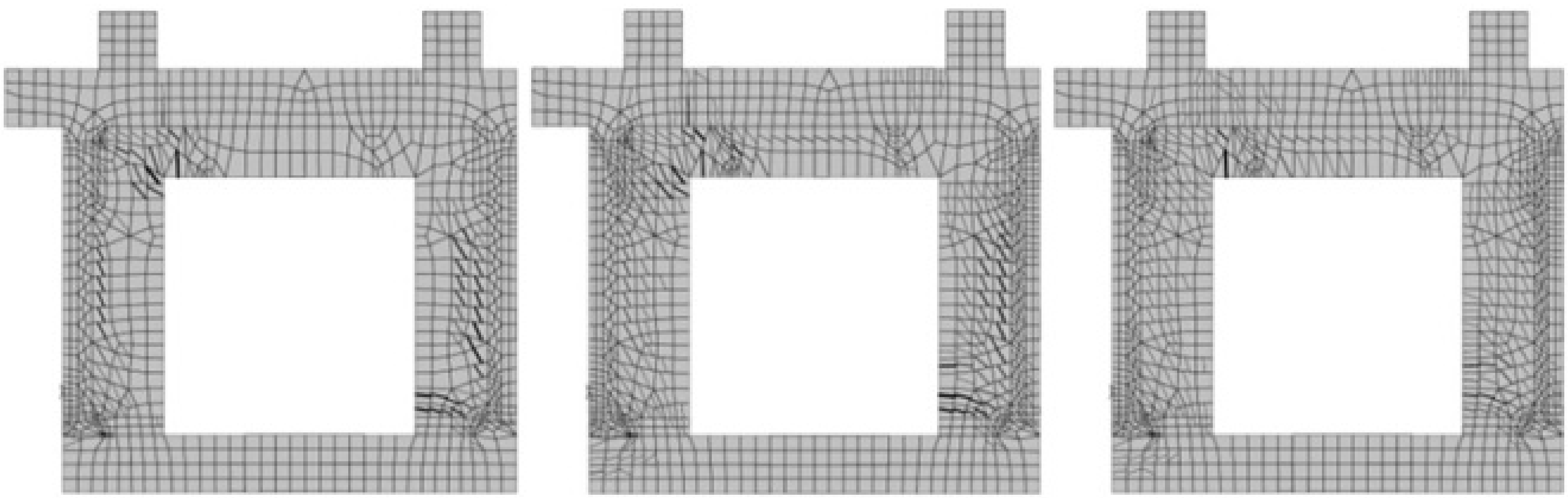
The load displacement diagram and the primary results of the tested and modelled wall with large door cut out are presented in these two figures.



PRIOR-TO-DAMAGE STRENGTHENED WALL WITH LARGE DOOR CUT-OUT RESULTS				
Element	First diagonal crack in both piers	FRP failure initiation	Maximum lateral load imposed	Drift level corresponding to max lateral load imposed
Tested	δ = 9.42 mm drift level	10.98 mm	339.8 kN	18.83 mm
Modelled	δ = 2.25 mm drift level	-	257 kN	5.16 mm

Prior-to-damage strengthened wall with large door cut-out

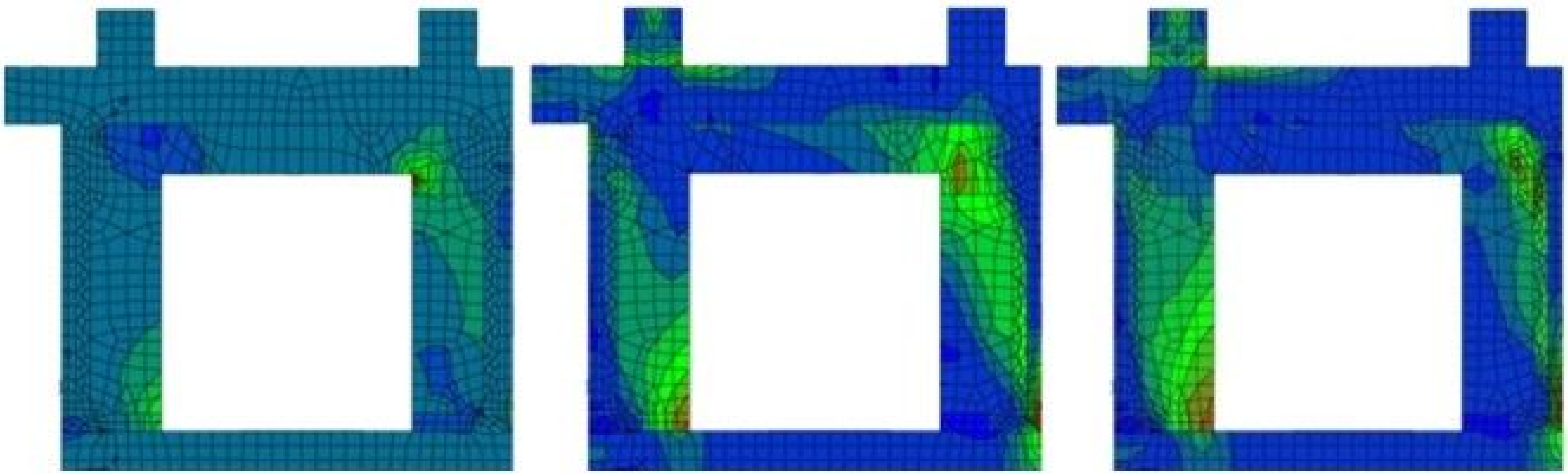
- **The figure below shows the crack state of the prior-to-damage strengthened wall with large door cut-out according to the numerical modelling.**



(a) (b) (c)

Crack state at (a) – 2.25 mm, (b) - 5.2 mm and (c) - 7.27 mm drift level

Prior-to-damage strengthened wall with large door cut-out



(a) (b) (c)

Compressive strain at (a) 5.2 mm, compressive stress at (b) 5.2 mm and (c) 7.27 mm.

Conclusion

- **The analysis predicted well the tendency of the experimental curves, sharing the same critical regions of the specimens.**
- **In terms of the maximum load supported by the element, the modelled wall with a narrow door cut-out recorded 541.8 kN while the tested specimen had 581.8 kN.**
- **The maximum drift level obtained by the modelled wall with a narrow door cut-out was 9.68 mm while for the tested specimen was 10.98 mm.**
- **Failure of the modelled wall with narrow door cut-out occurred due to concrete crushing in pier 2 and for the modelled post-damage strengthened wall with narrow door cut-out due to FRP reinforcement yielding at the left upper corner of the opening and concrete crushing in pier 2.**

Conclusion

- **In terms of the maximum load supported by the element, the modelled wall with large door cut-out recorded 303 kN while the tested specimen had 317.7 kN.**
- **The modelled prior-to-damage strengthened wall with large door cutout recorded 257 kN and the tested specimen had 339.8 kN.**
- **Concluding upon the performance of the strengthening system this was efficient from the point of view of the load bearing capacity.**
- **Also from the recorded data available one can remark that the loss of bearing capacity caused by a door cut-out with respect to the solid wall is more than significant.**

TEXTILE REINFORCED MORTAR STRENGTHENING OF A PRECAST REINFORCED CONCRETE WALL PANEL USING CARBON FIBER GRID

Carla TODUȚ
PhD Student

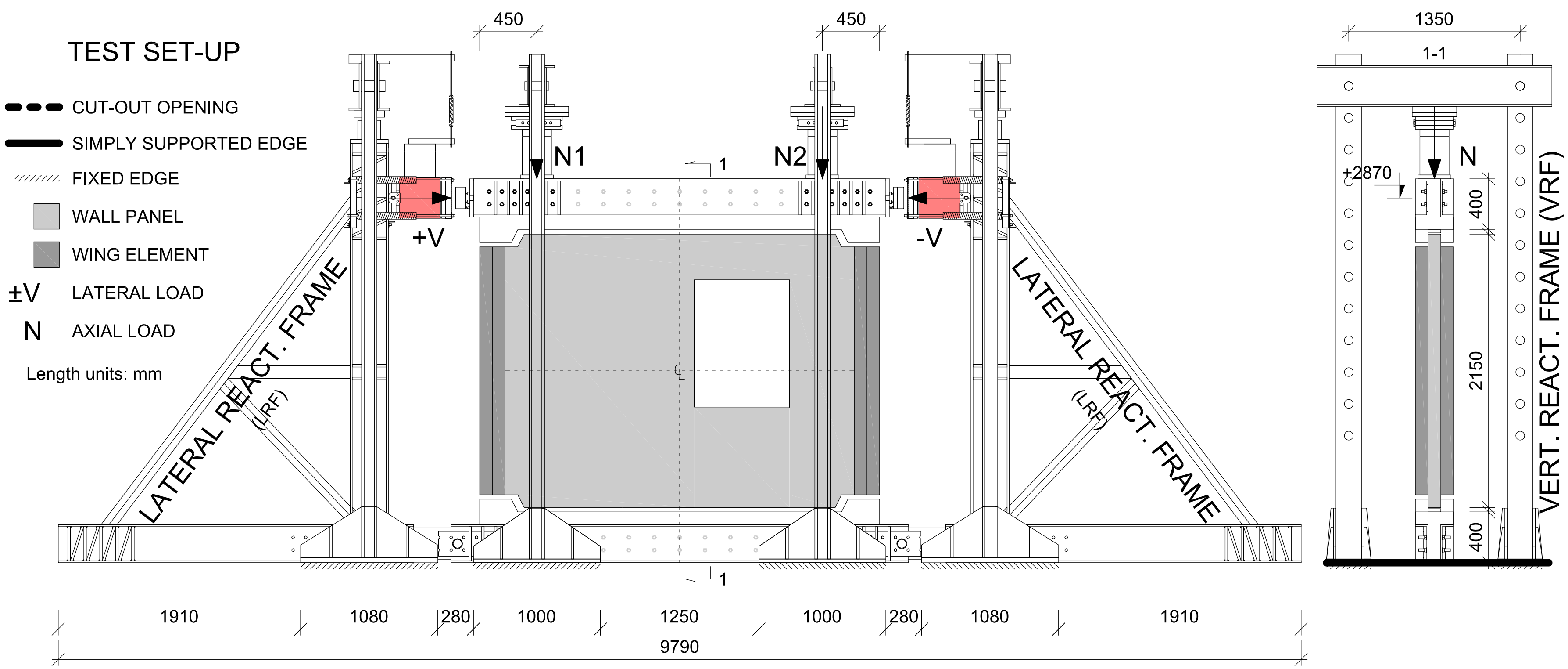
Valeriu STOIAN
PhD, Professor

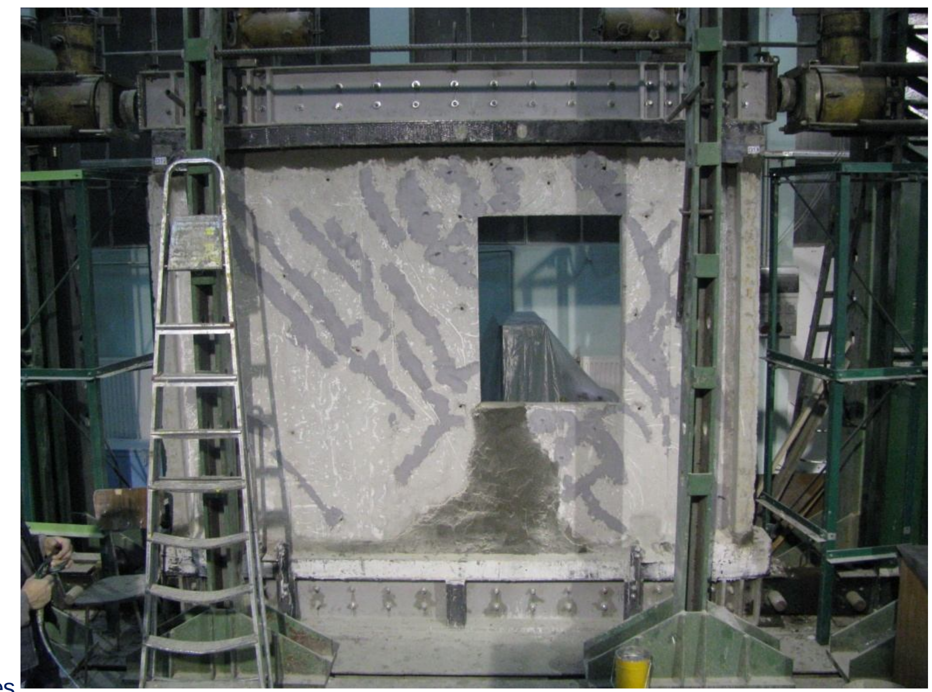
István DEMETER
PhD, Assistant

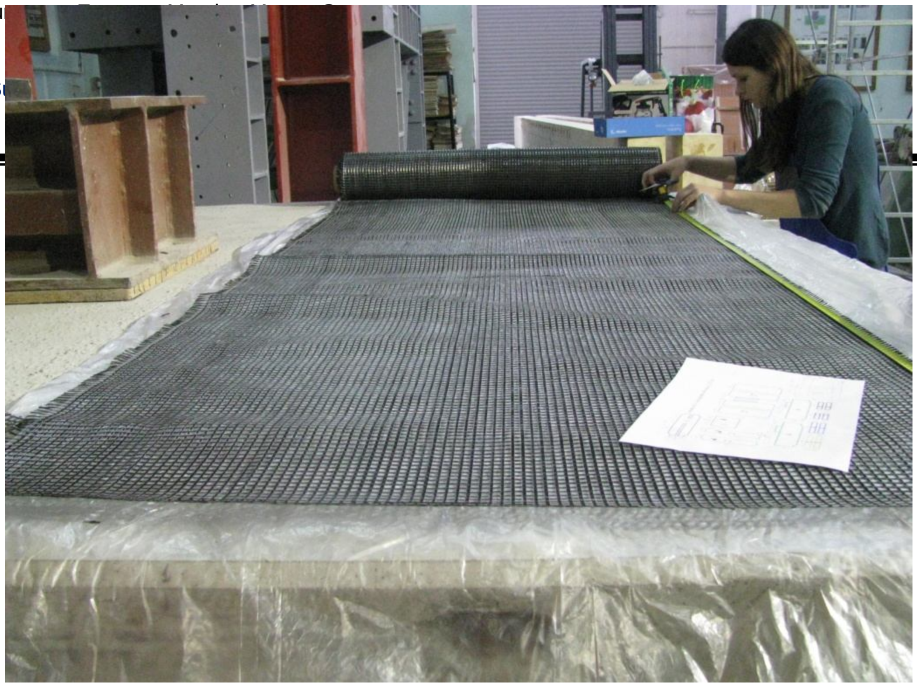
Mihai FOFIU
PhD Student

Politehnica University of Timișoara

- Large number of PRCLWP buildings across the country
- Interventions on them become more and more necessary after 50 years of existence
- Determine the seismic performance of the PRCWP
- Investigate the weakening induced by cut-outs in walls due to various reasons.
- Analyse various strengthening systems for initial load bearing capacity restoration of the elements
- Evaluate the material and labour costs for each strengthening system since few literature is known on this aspect

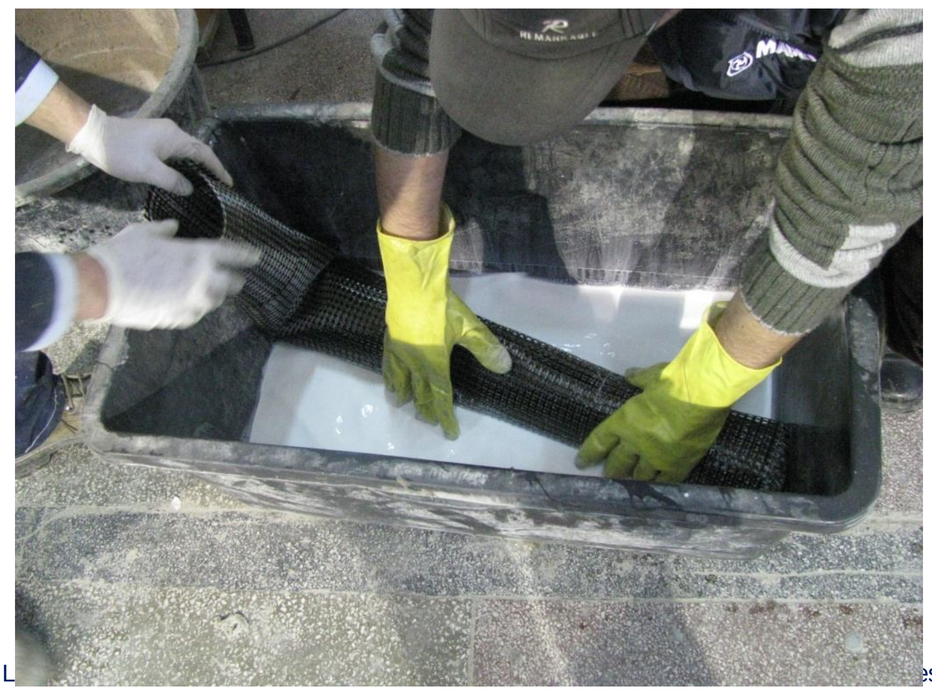

TEST SET-UP
CUT-OUT OPENING
SIMPLY SUPPORTED EDGE
FIXED EDGE
WALL PANEL
WING ELEMENT
±V LATERAL LOAD
N AXIAL LOAD
Length units: mm
450
450
N1
N2
1
+V
-V
LATERAL REACT. FRAME (LRF)
LATERAL REACT. FRAME (LRF)
1910
1080
280
1000
1250
1000
280
1080
1910
9790
1350
1-1
N
+2870
400
2150
400
VERT. REACT. FRAME (VRF)

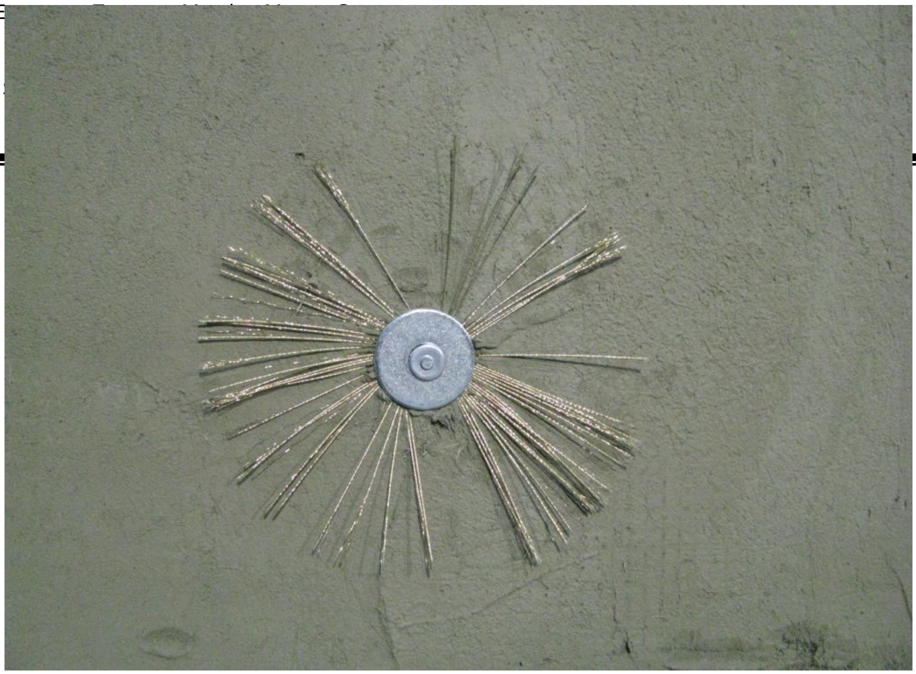
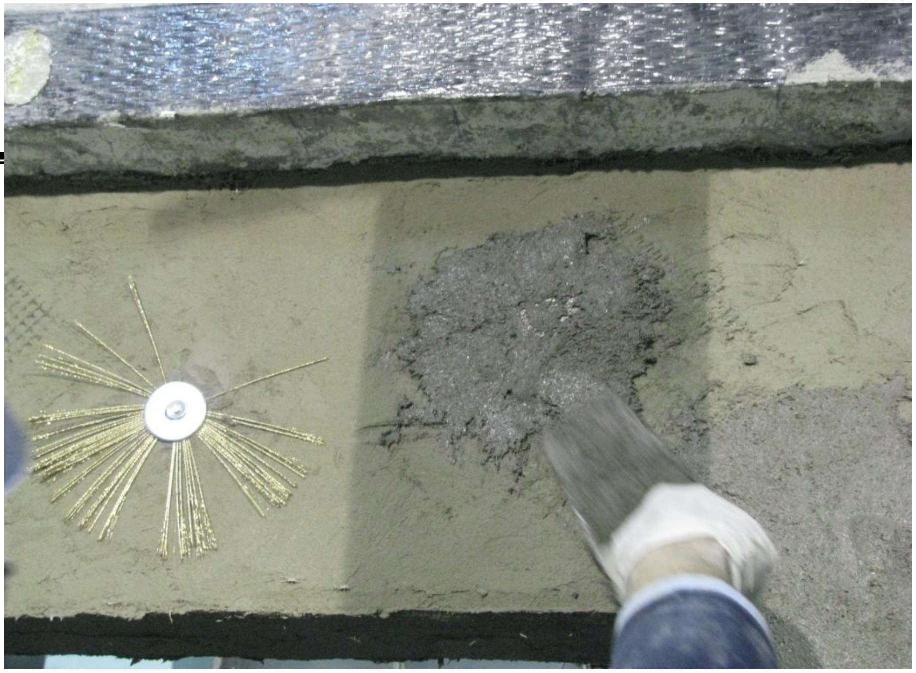




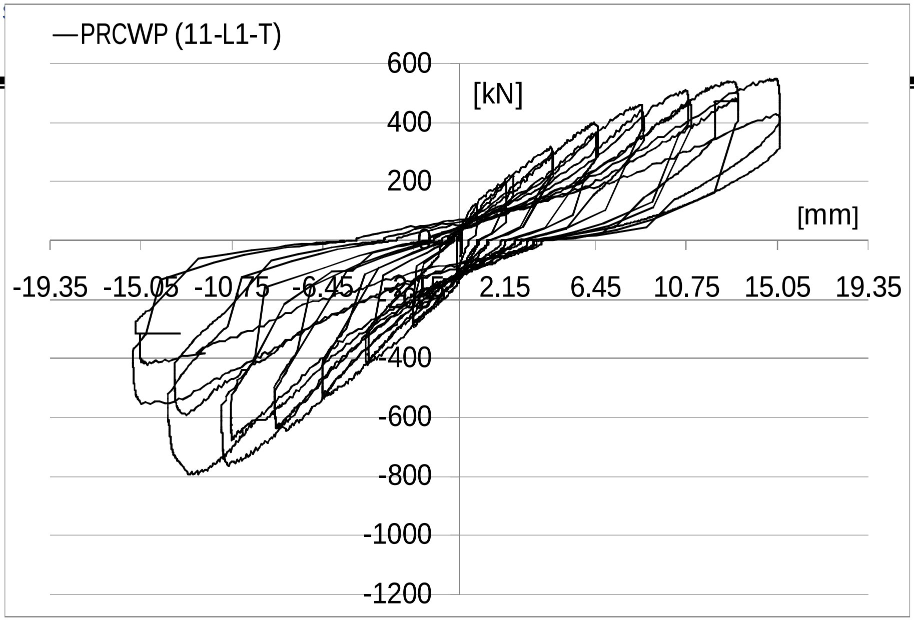

Easy for you

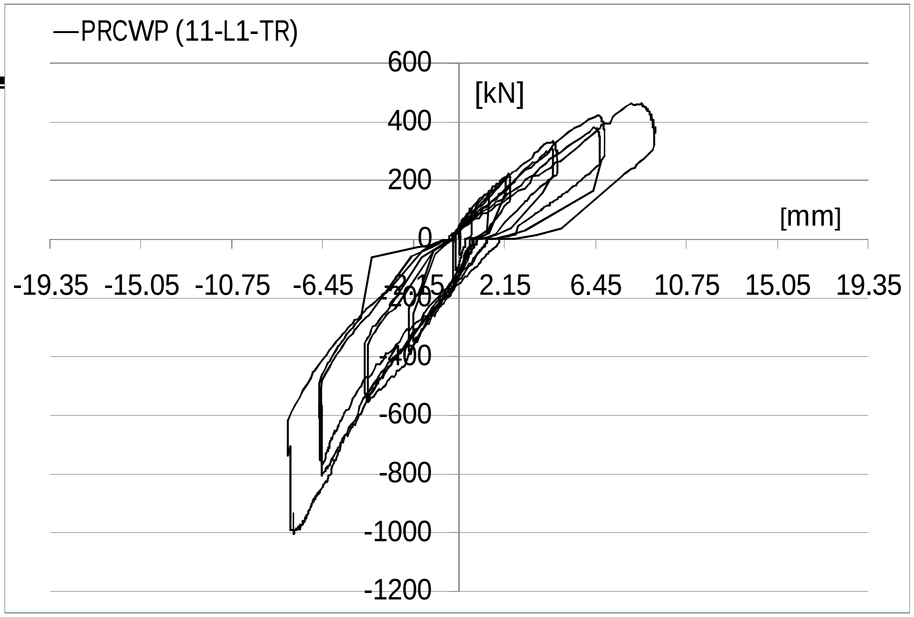

MAPEI

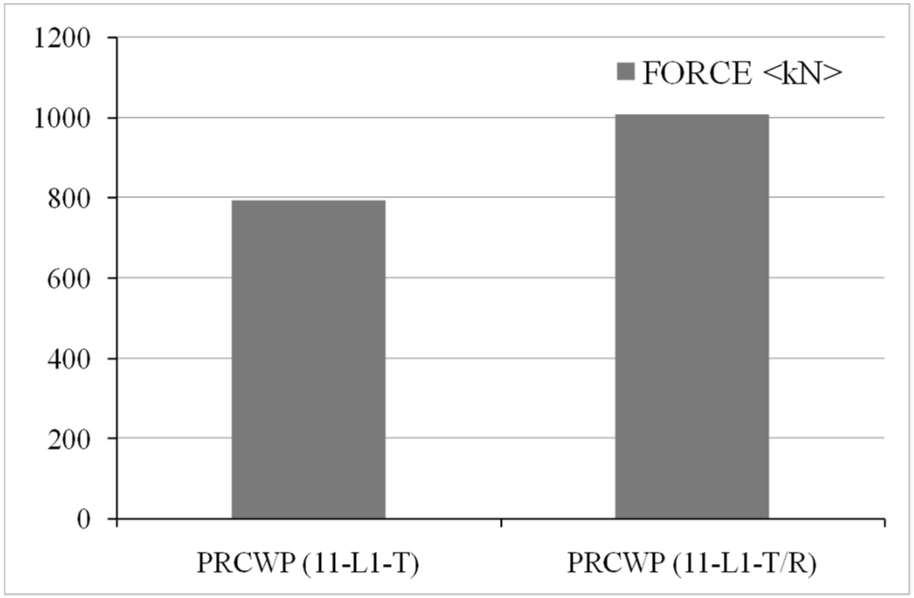


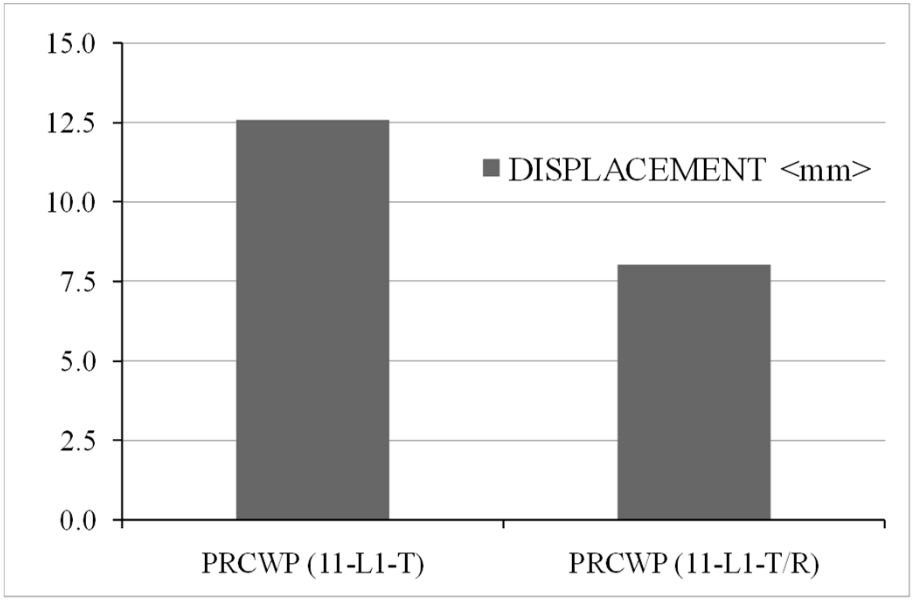




2
PRCWP DRIFT
11-L1-T/R 8.6 mm


—PRCWP (11-L1-T)
[kN]
[mm]
600
400
200
0
-200
-400
-600
-800
-1000
-1200
-19.35 -15.05 -10.75 -6.45 -2.15 2.15 6.45 10.75 15.05 19.35

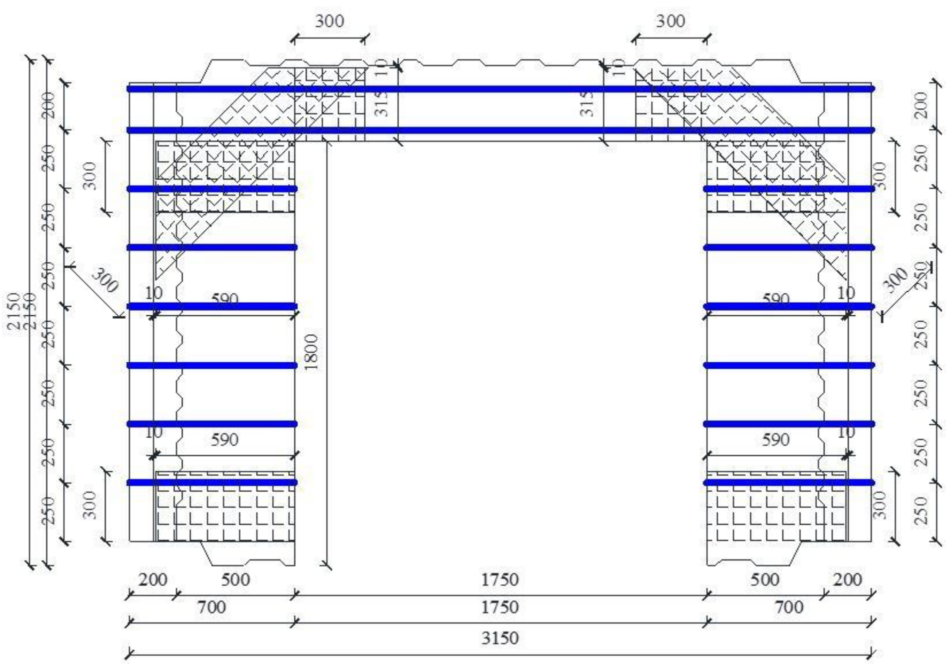

—PRCWP (11-L1-TR)
[kN]
[mm]
600
400
200
0
-200
-400
-600
-800
-1000
-1200
-19.35 -15.05 -10.75 -6.45 -2.15 2.15 6.45 10.75 15.05 19.35


FORCE <kN>
1200
1000
800
600
400
200
0
PRCWP (11-L1-T)
PRCWP (11-L1-T/R)

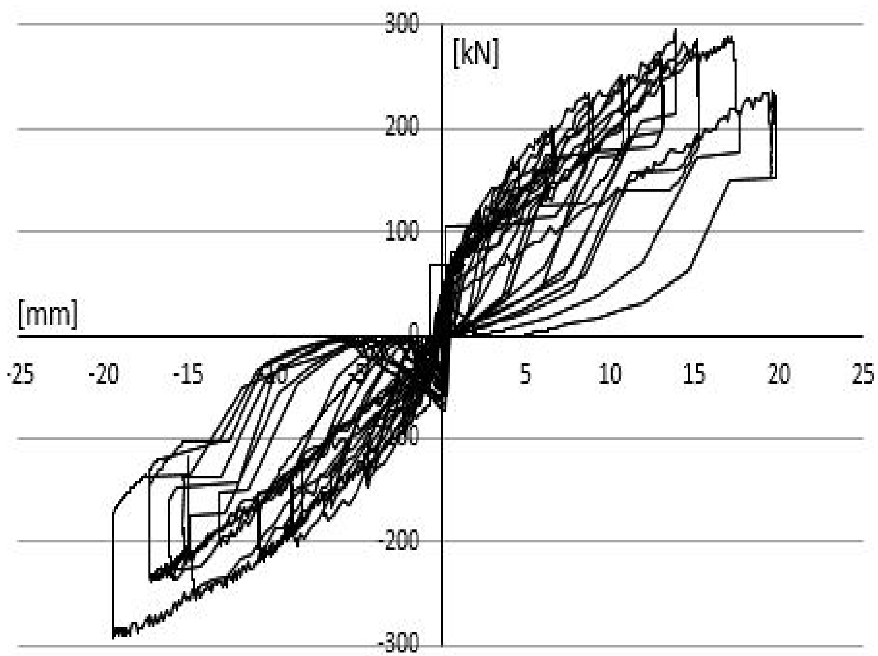

DISPLACEMENT <mm>
15.0
12.5
10.0
7.5
5.0
2.5
0.0
PRCWP (11-L1-T)
PRCWP (11-L1-T/R)

- In terms of maximum load supported by the element the PRCWP (11-L1-T) recorded 793.5 kN , while PRCWP (11-L1-T/R) 1007.5 kN.
- Drift level corresponding to the maximum load was 12.59 mm for the unstrengthened wall while for the post-damage strengthened one 8.02 mm.
- The PRCWP (11-L1-T/R) could not be taken to failure due to the available capacity of the testing facility, but analyzing the data one can remark that at a displacement level of 8.02 mm we have an increase in load bearing capacity of 60%.
- The strengthening system using TRM with CF grid proved to be the most expensive in the current experimental program (276.42 EUR/ m^2), but we have to take into consideration the fact that the crack injection was not performed in the other cases and also the idea of strengthening versus retrofitting implying the carbon fiber grid wraps raised the total price.

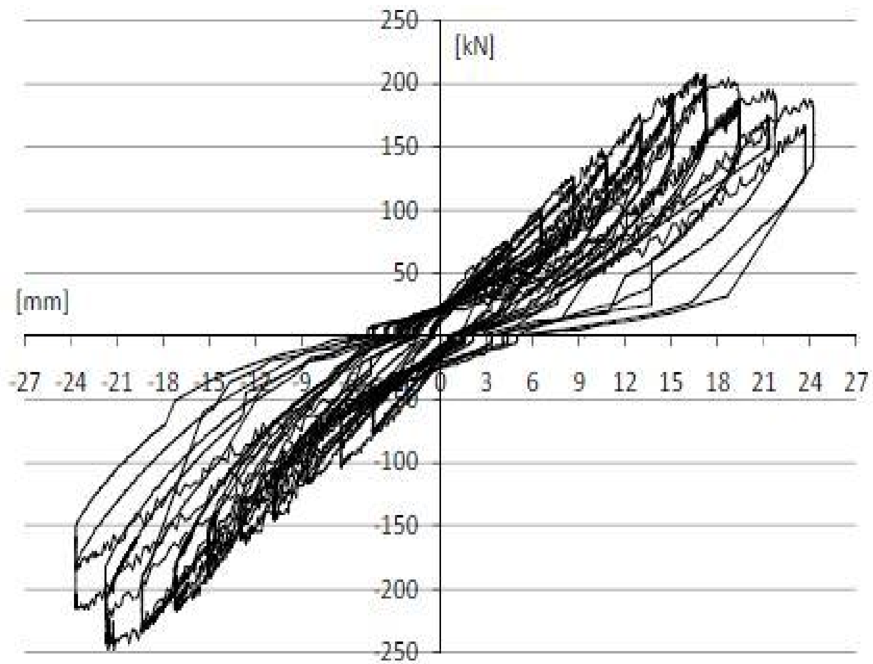
Prior to damage strengthened wall with large door cut-out using mixed NSM-EBR technique results



Force displacement diagram comparison



Strenghten



Unstrenghten

Slide 283

A1 Author, 6/18/2014

CONCLUSIONS

- The objective was to obtain similar bearing capacity as a PRCWP with an initial large door opening
- From the results it can be clearly seen that the retrofitting strategy was a success
- The bearing capacity was increased with 17 % from 246.5 kN to 288 kN
- The drift was 8.4 % smaller on the retrofitted specimen

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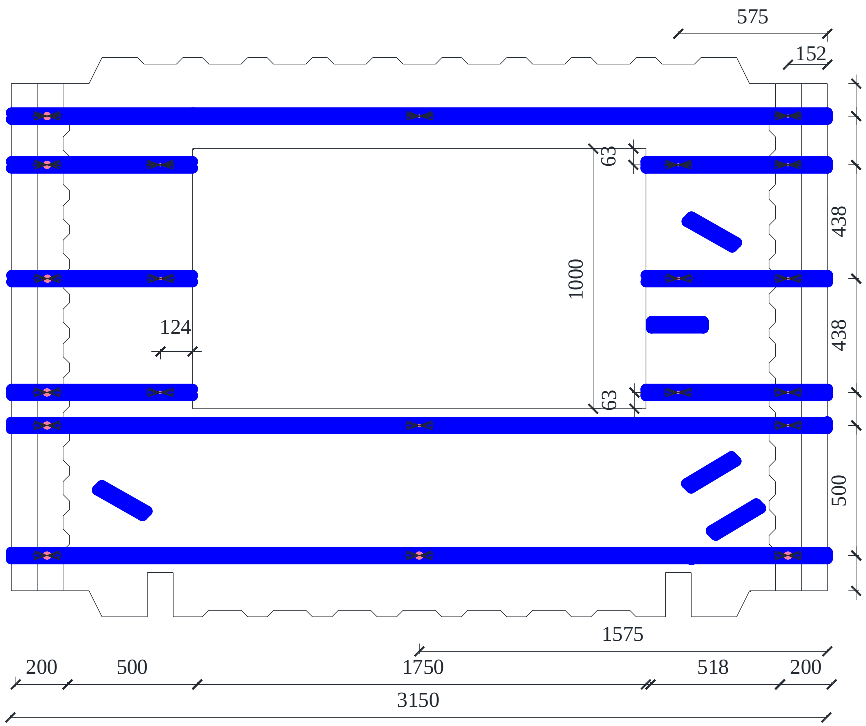
A1 Author, 6/18/2014

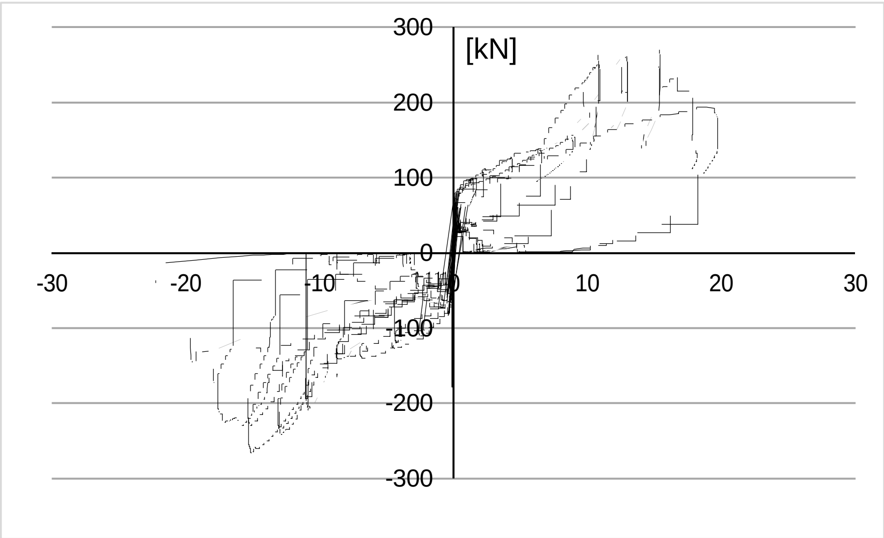
Post-damage strengthened wall using EBR strips



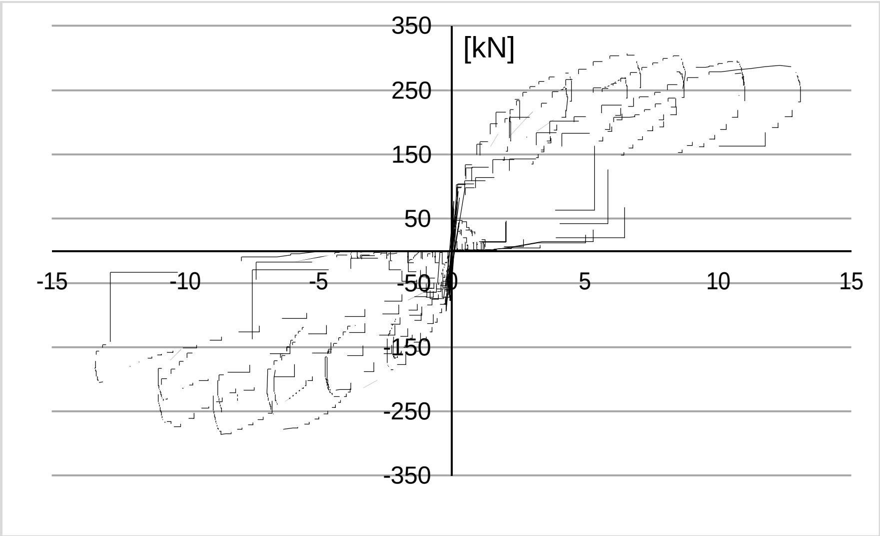
Slide 285

A2 Author, 6/18/2014

Force displacement diagram comparison



Strenghten



Unstrenghten

Slide 286

A2 Author, 6/18/2014

CONCLUSIONS

- The objective was to reestalish the load bearing capacity of a PRCWP with an initial large window opening
- From the results it can be clearly seen that the retrofitting strategy was a success
- The retrofitting technique was able to reestablish 90,2% of the initial load bearing capacityThe drift was 8.4 % smaller on the retrofitted specimen

Slide 287

A2 Author, 6/18/2014

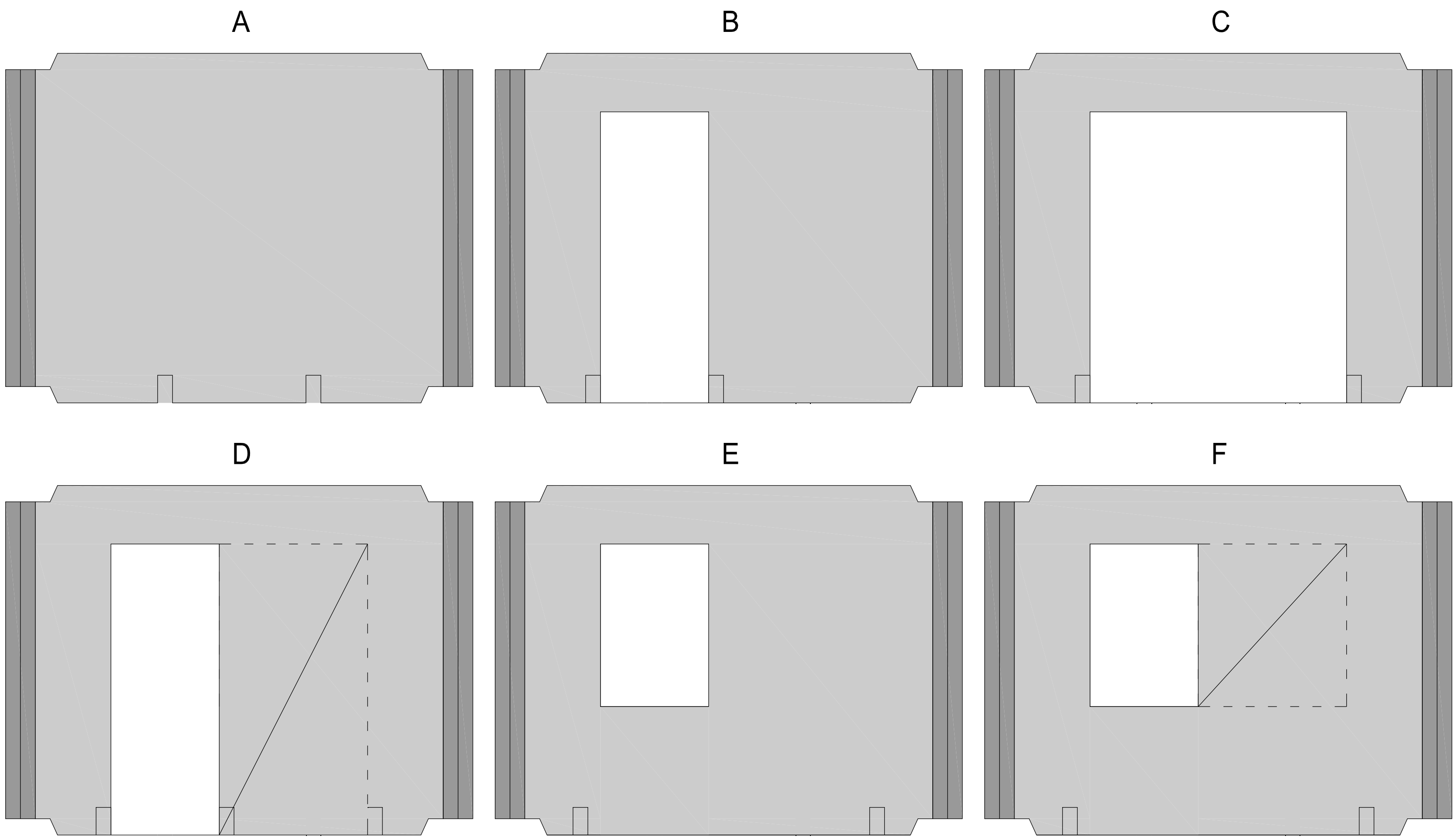
SUSCOS

STRUCTURAL REHABILITATION OF PRECAST REINFORCED CONCRETE WALL PANELS USING CFRP COMPOSITES

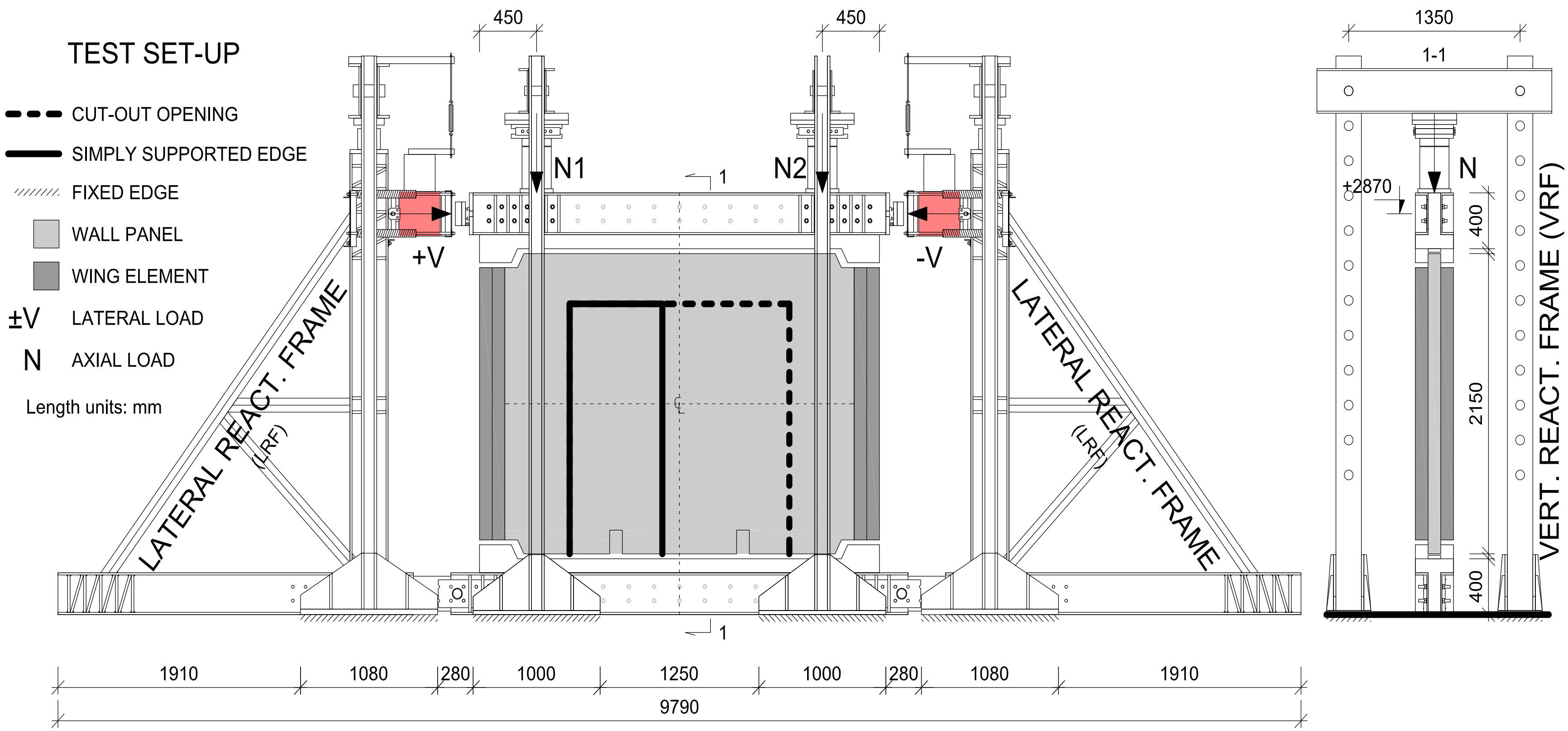
Carla Toduţ, Mihai Fofiu

Politehnica University of Timișoara

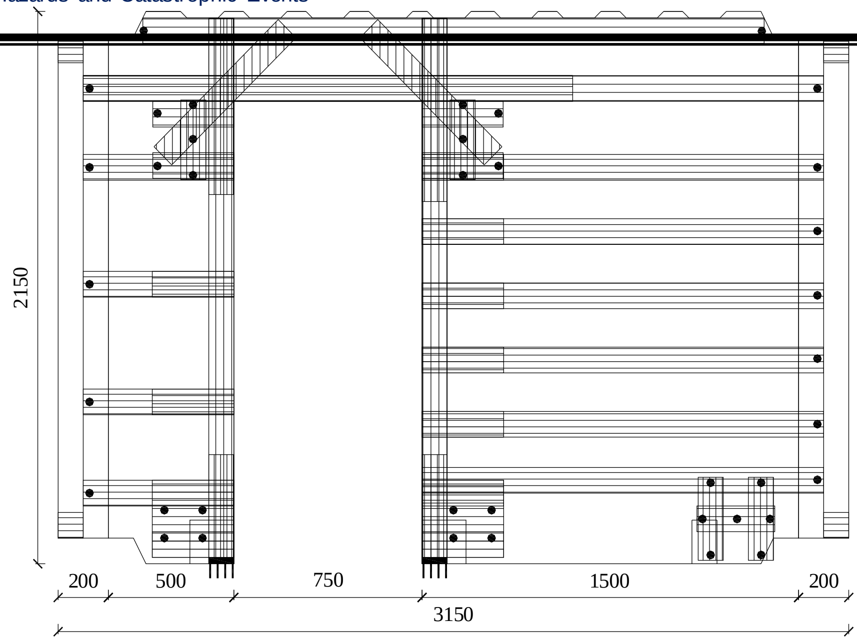
1. Motivation

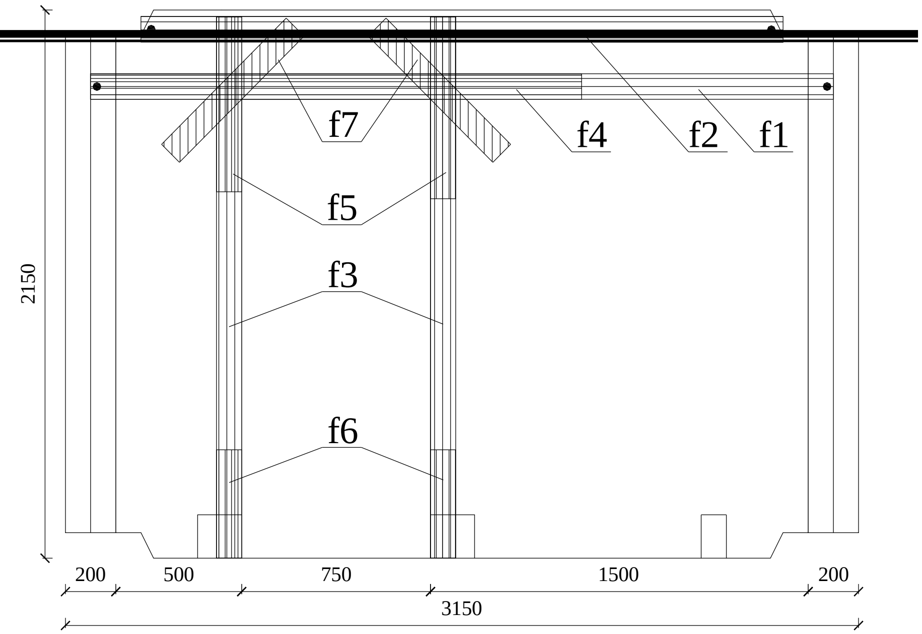


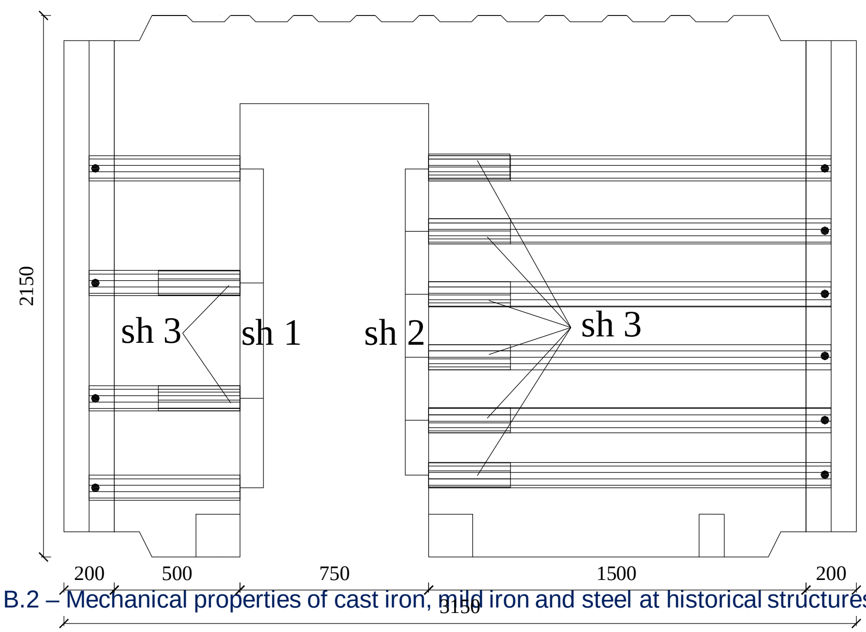
2. Experimental program description

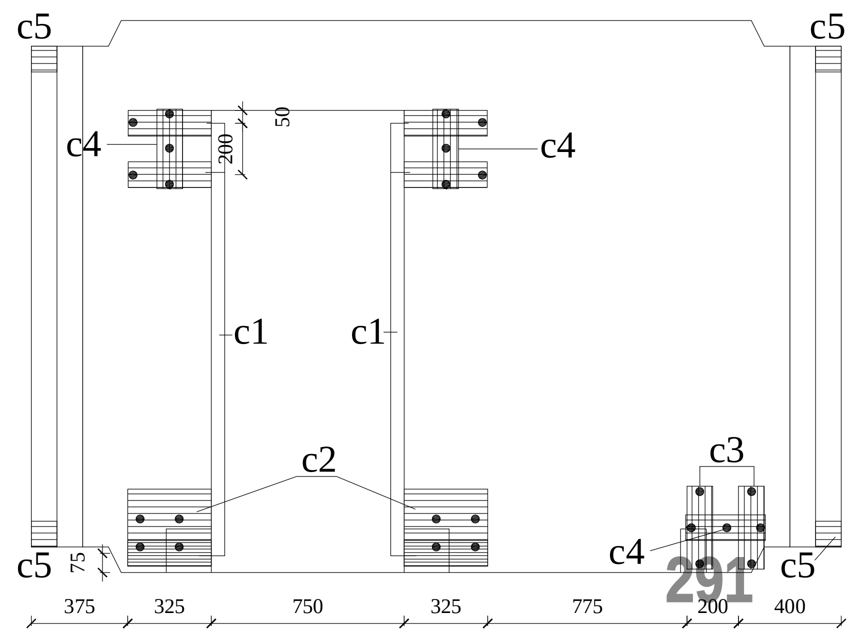


4. Strengthening strategy for specimen (7-E1-T/R)

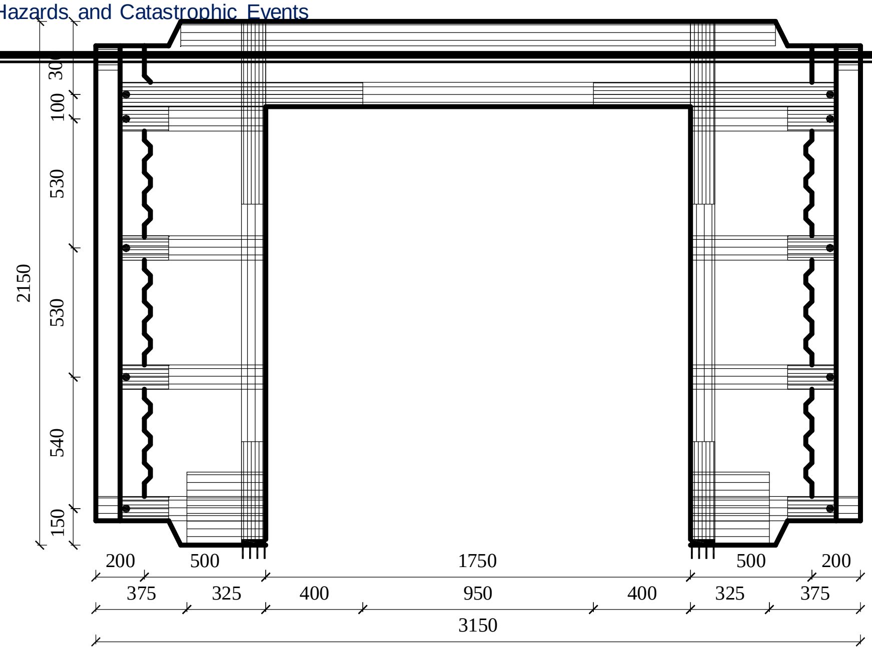


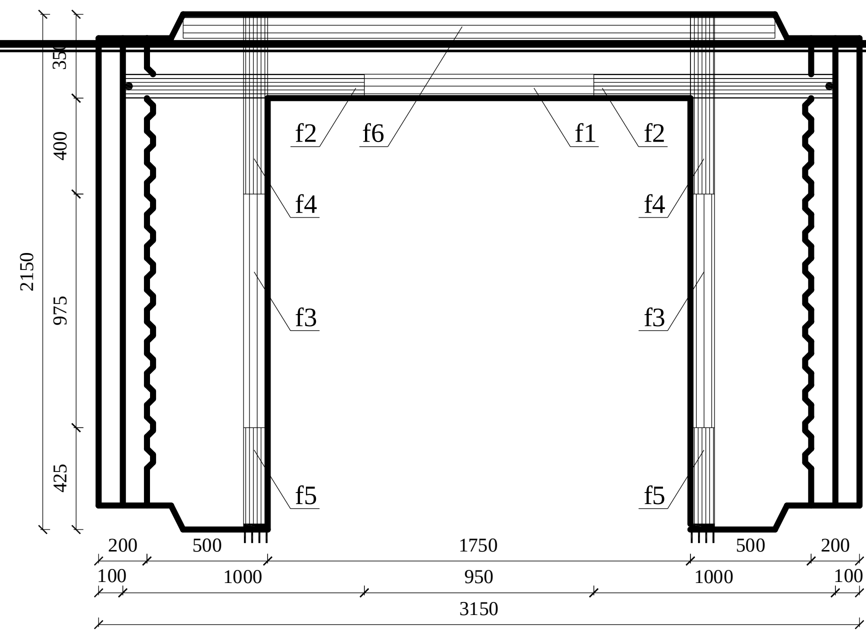


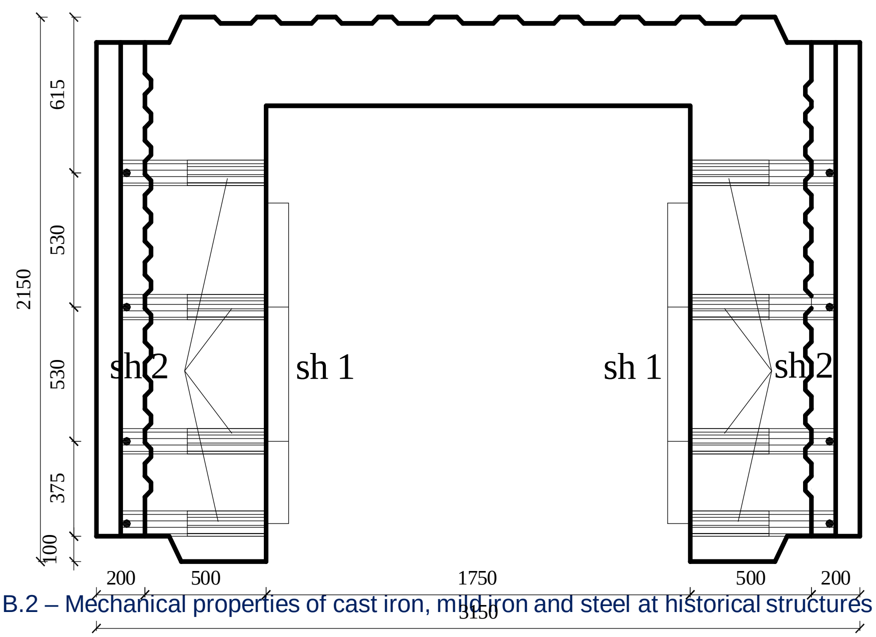


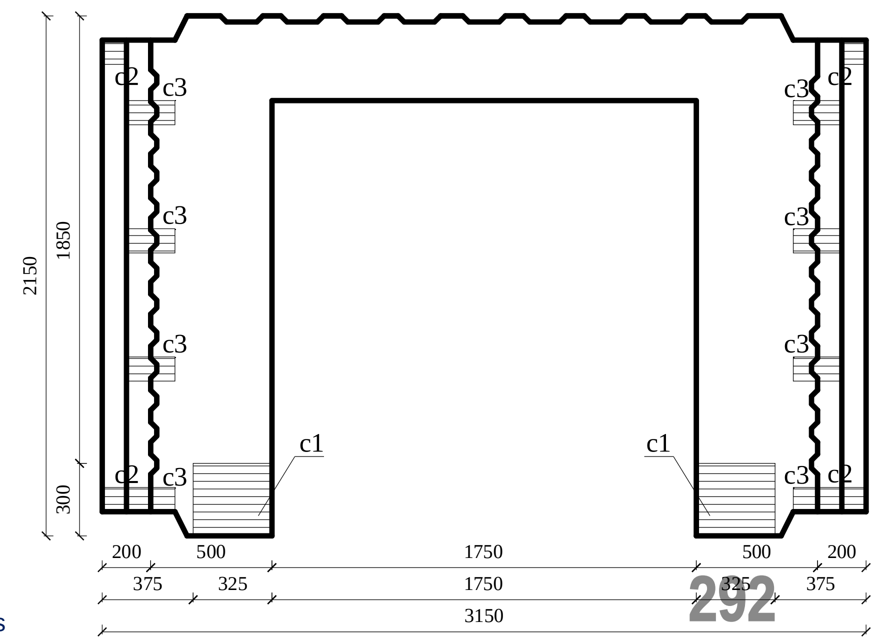


4. Strengthening strategy for specimen (8-E3-T/R)

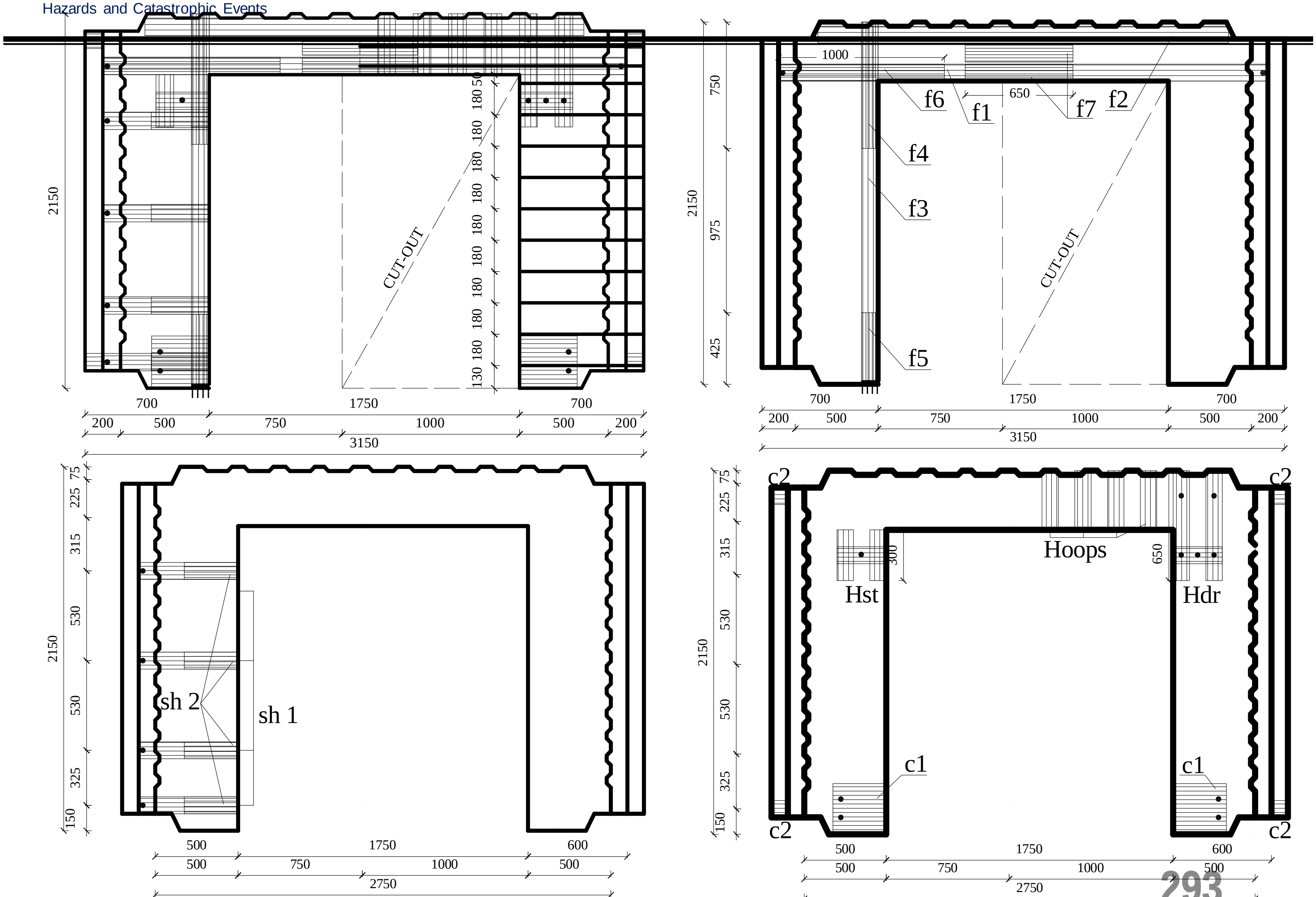


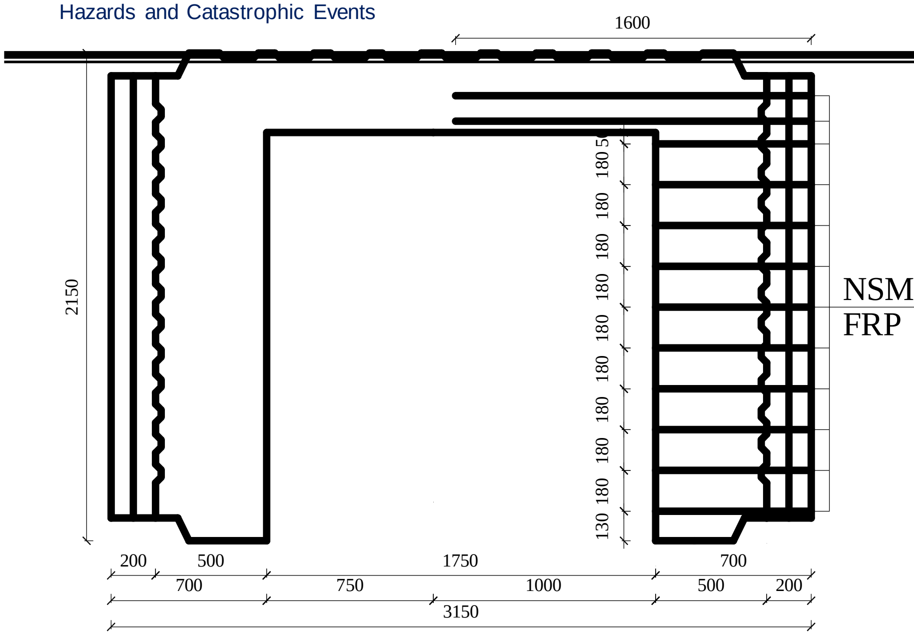




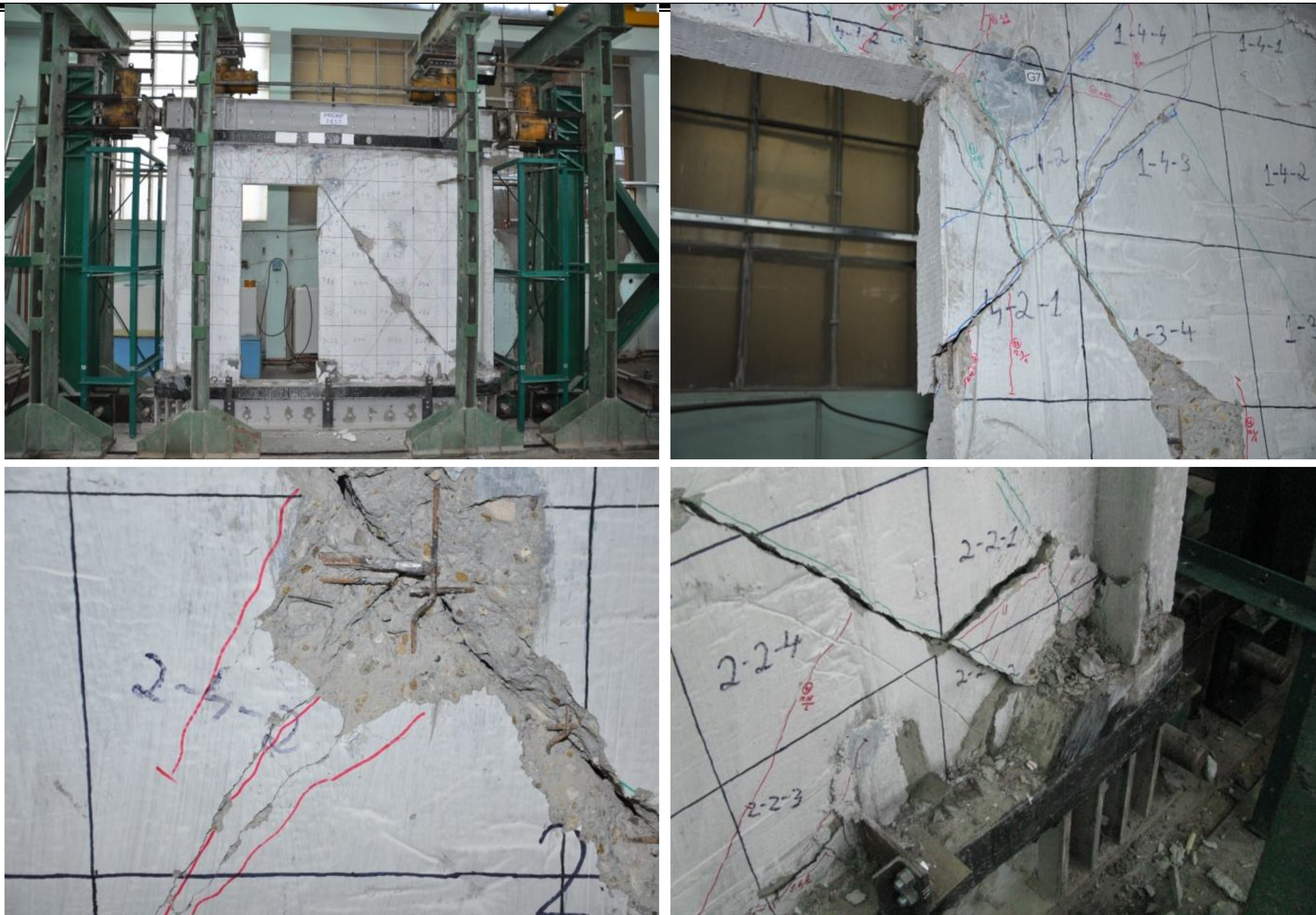


4. Strengthening strategy for specimen (9-E1/E3-R/T)




1600
2150
NSM
FRP
50
180
180
180
180
180
180
180
180
180
180
130
200
500
1750
700
700
750
1000
500
200
3150

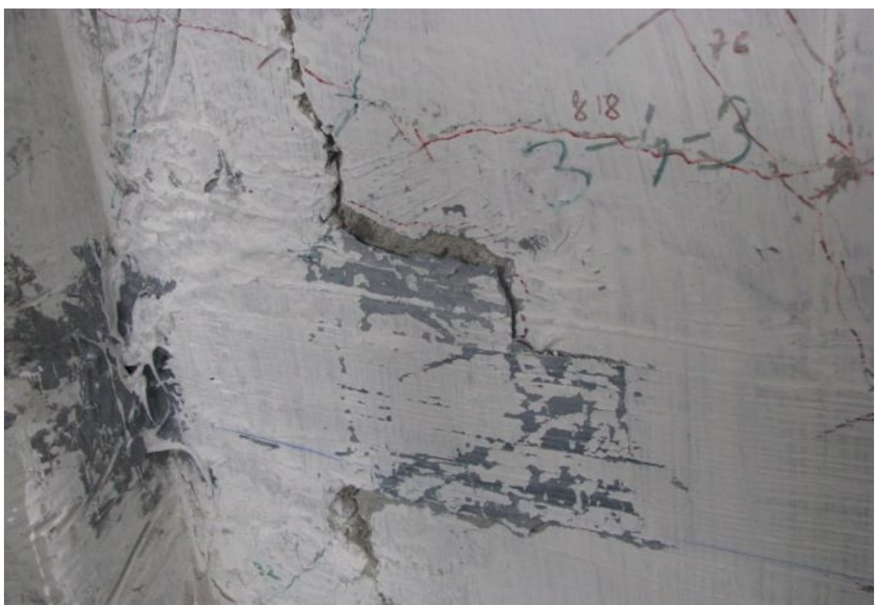
5. Failure details (PRCWP-7-E1-T)

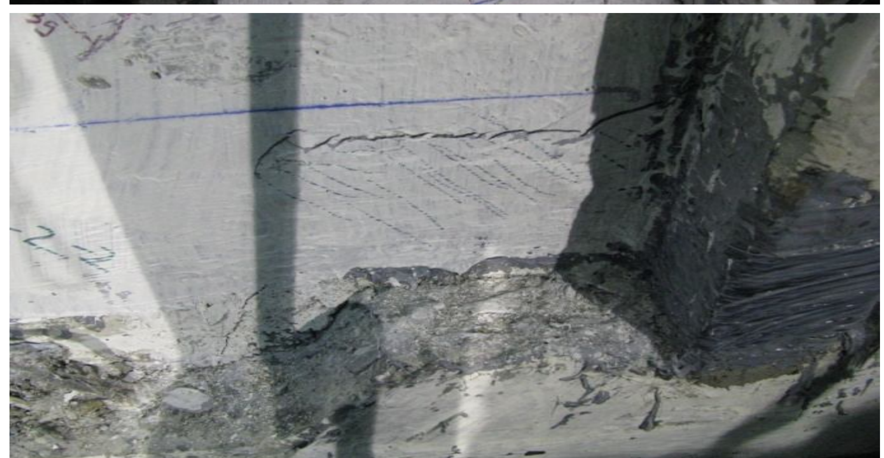


5. Failure details(PRCWP-7-E1-T/R)

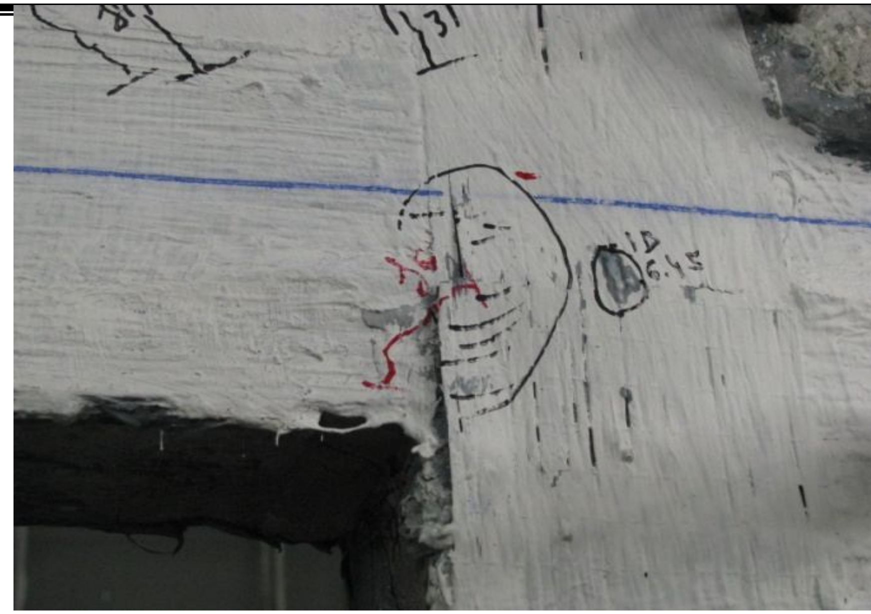
5. Failure details(PRCWP-8-E3-T)

5. Failure details(PRCWP-8-E3-T/R)

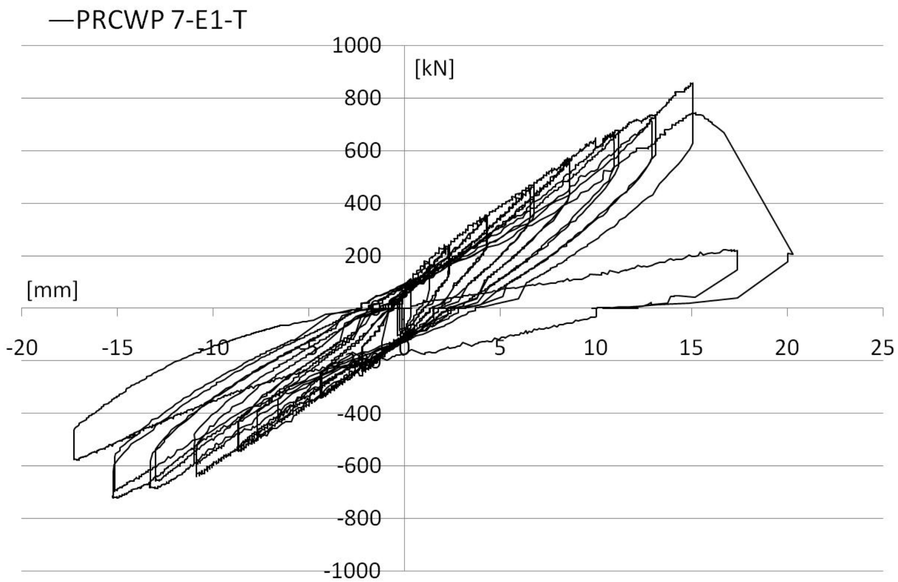


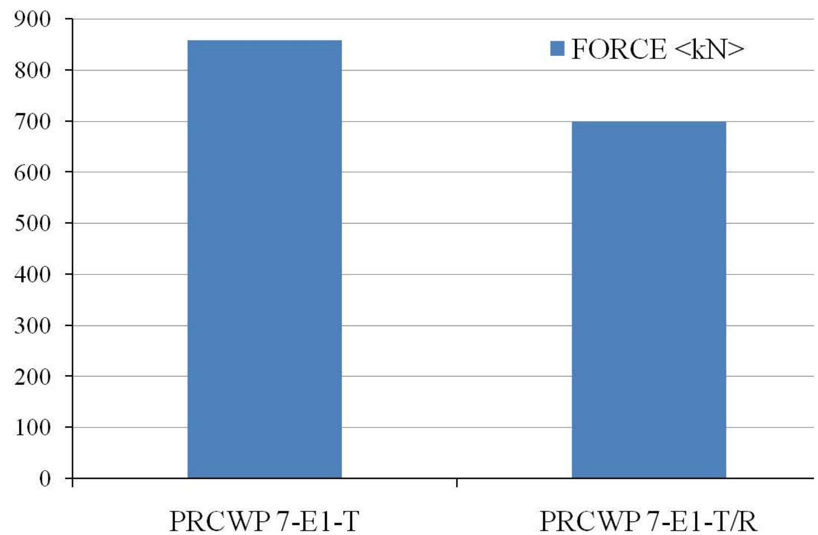


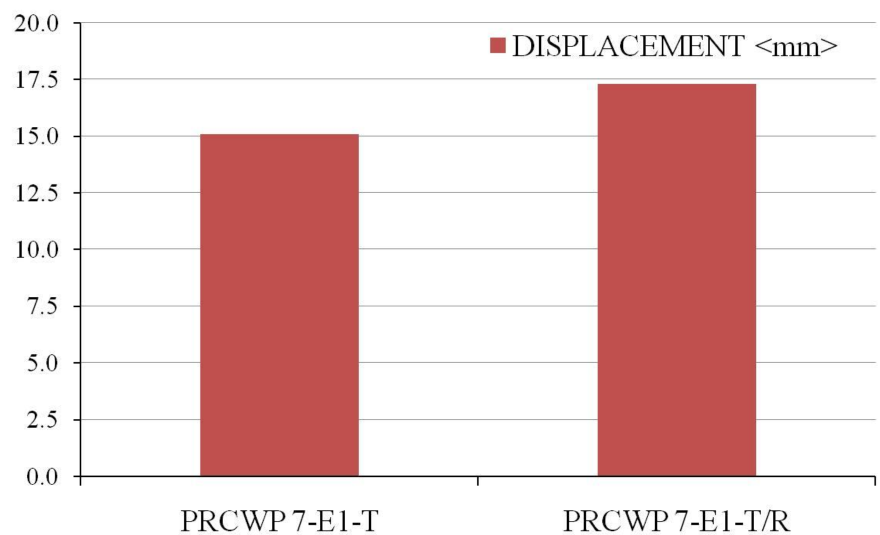
5. Failure details(PRCWP-9-E1/E3-R/T)

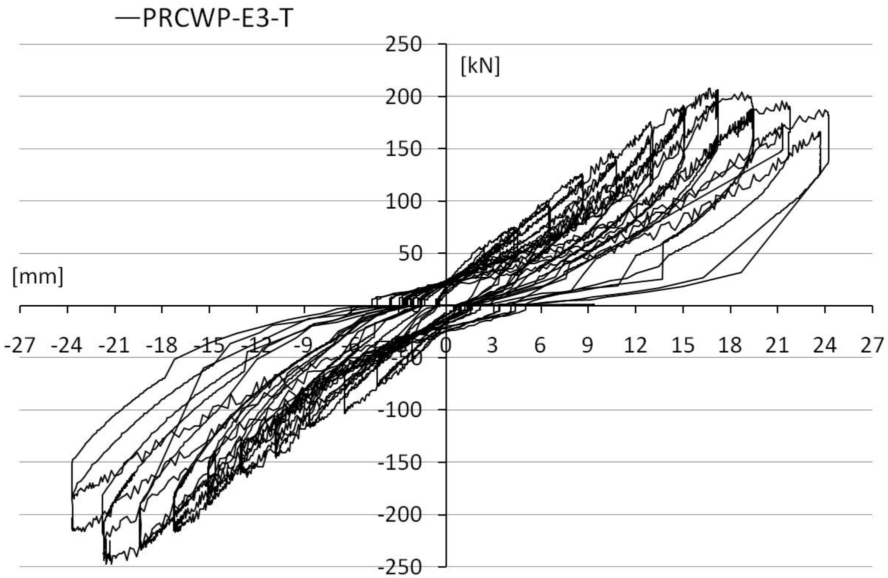
6. Results (PRCWP 7-E1-T & 7-E1-T/R)

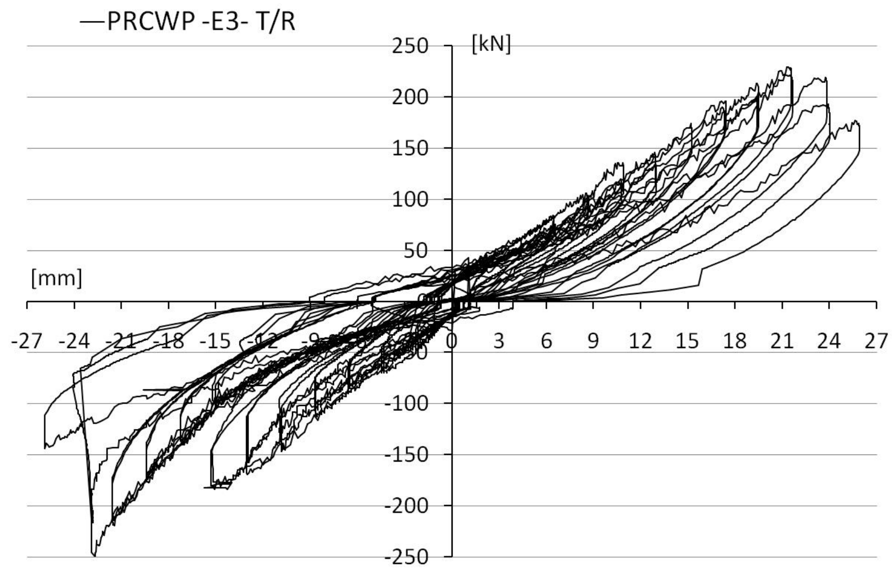


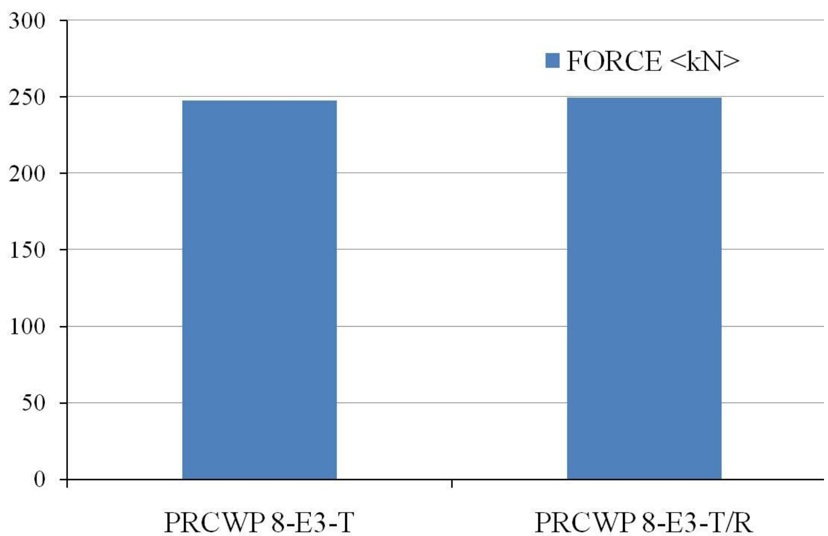


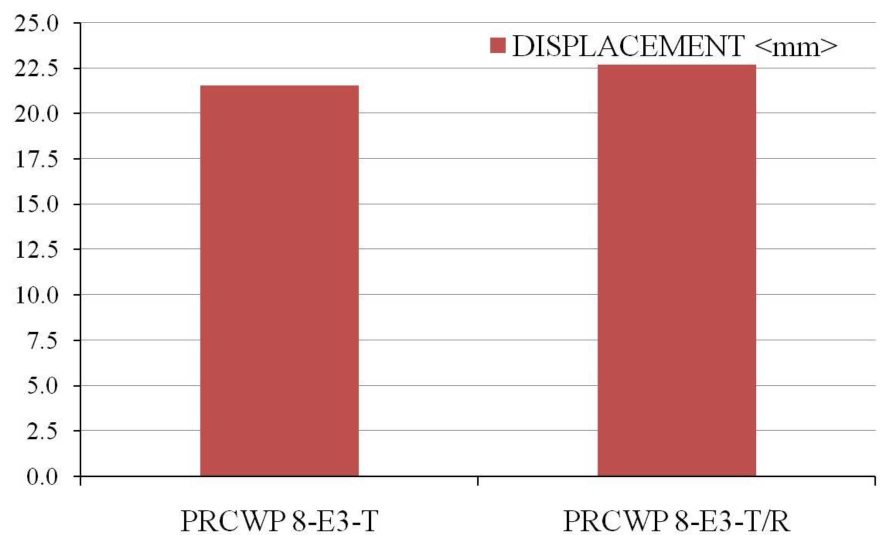


6. Results (PRCWP 8-E3-T & 8-E3-T/R)

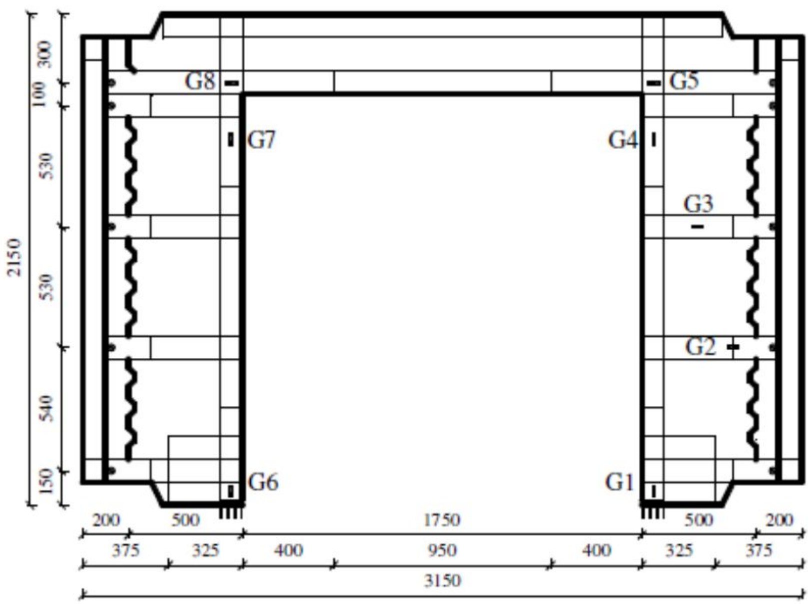


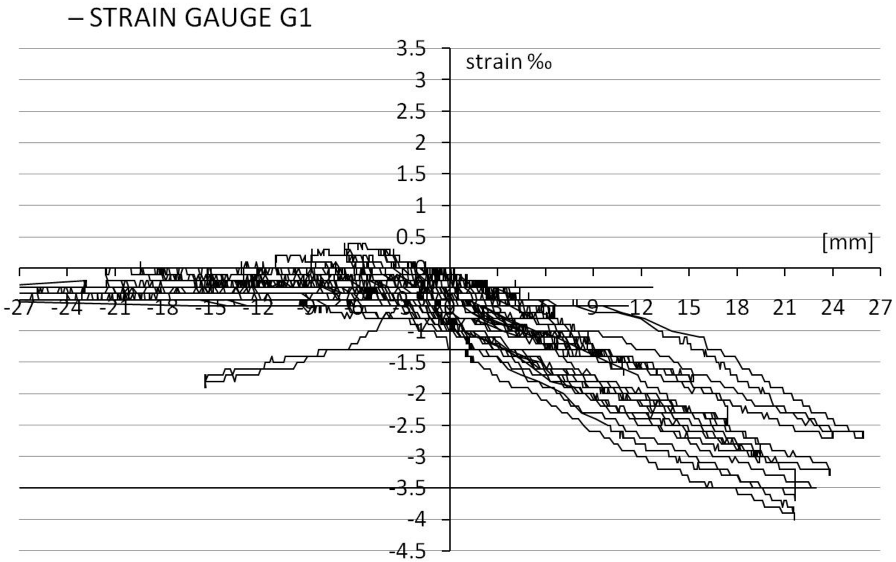


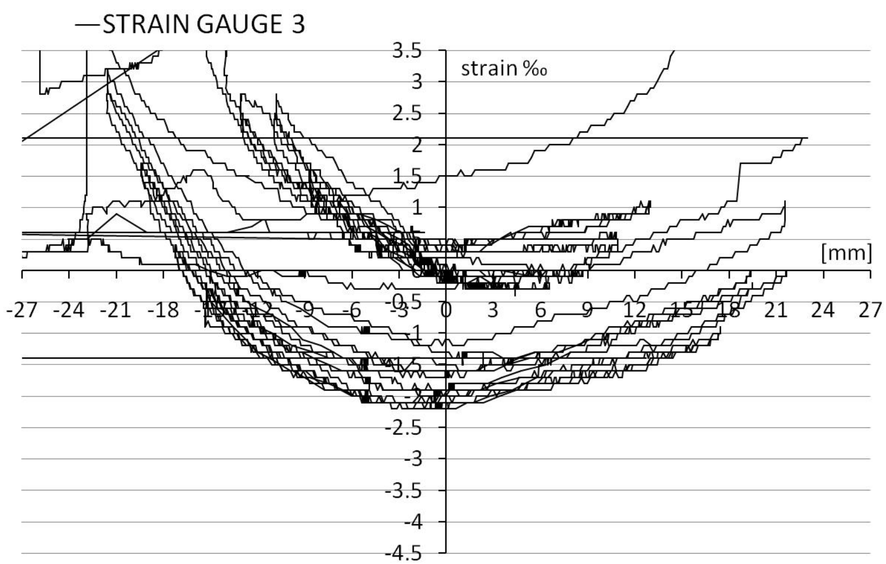




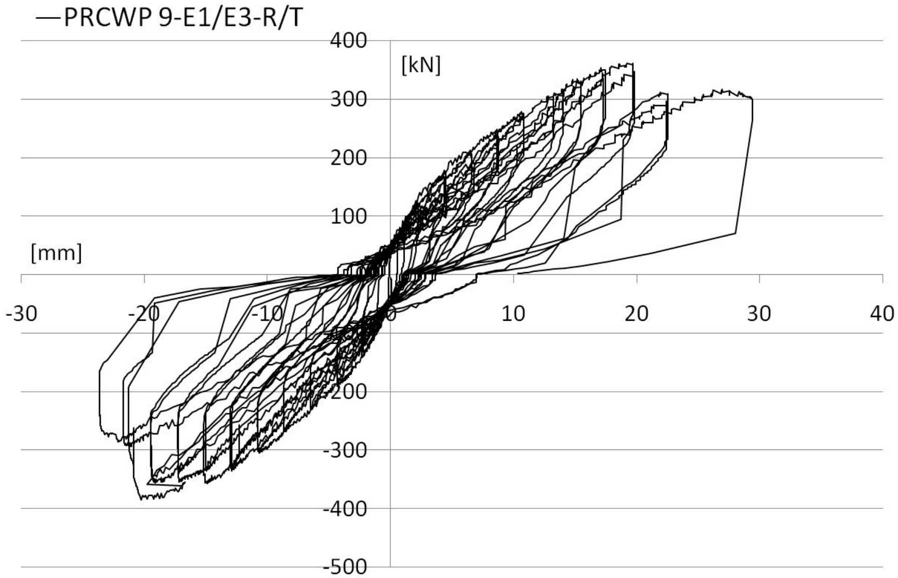
6. Results (PRCWP 8-E3-T & 8-E3-T/R)

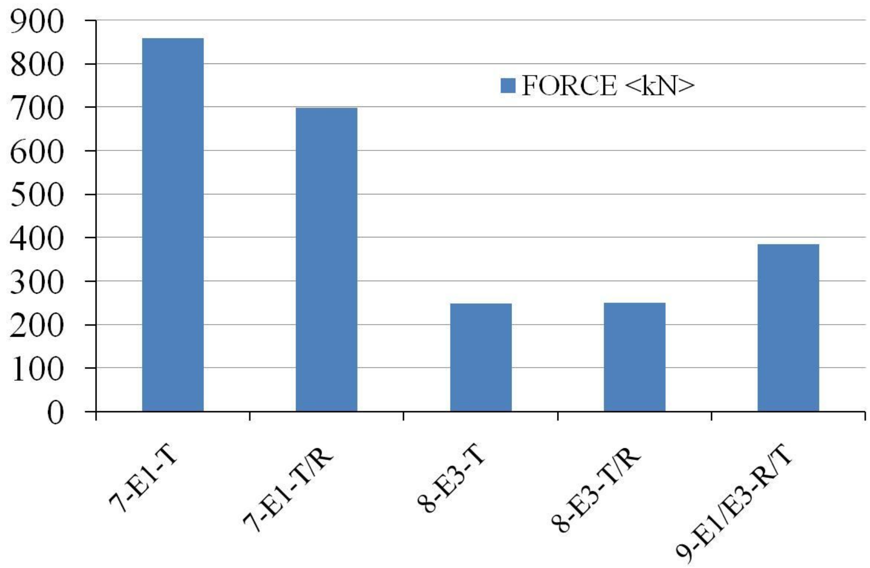


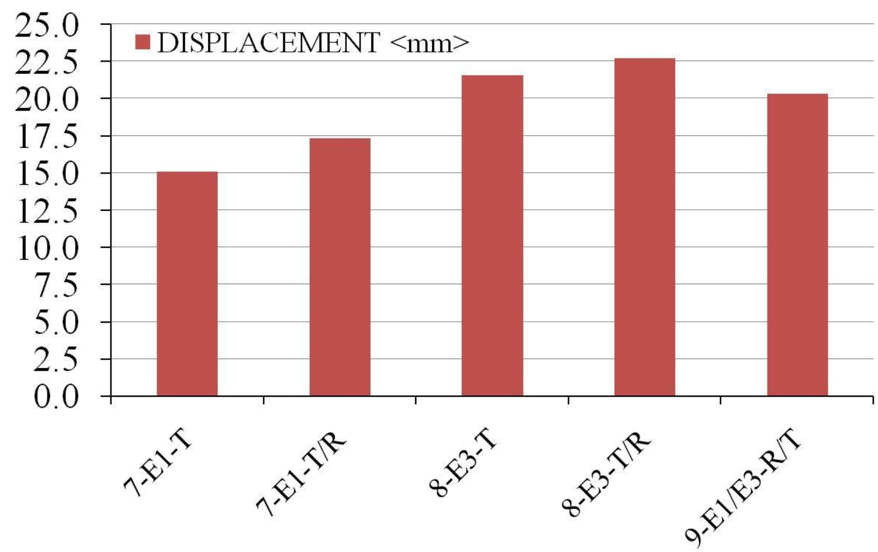




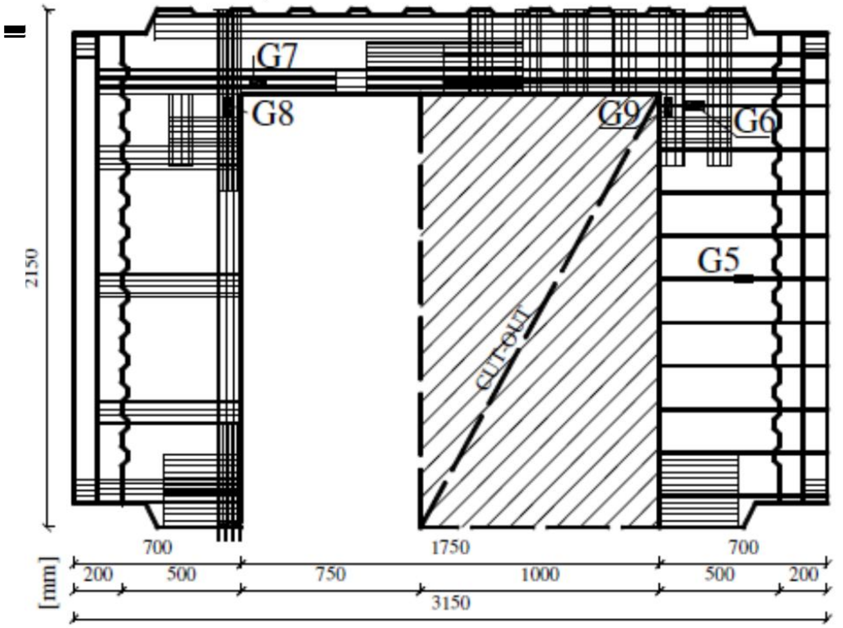
6. Results (PRCWP 9-E1/E3-R/T)

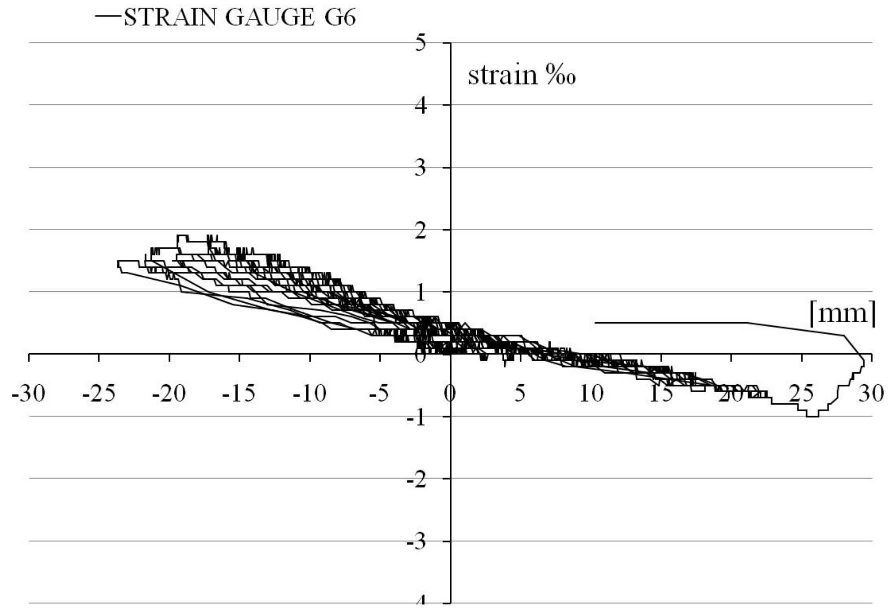


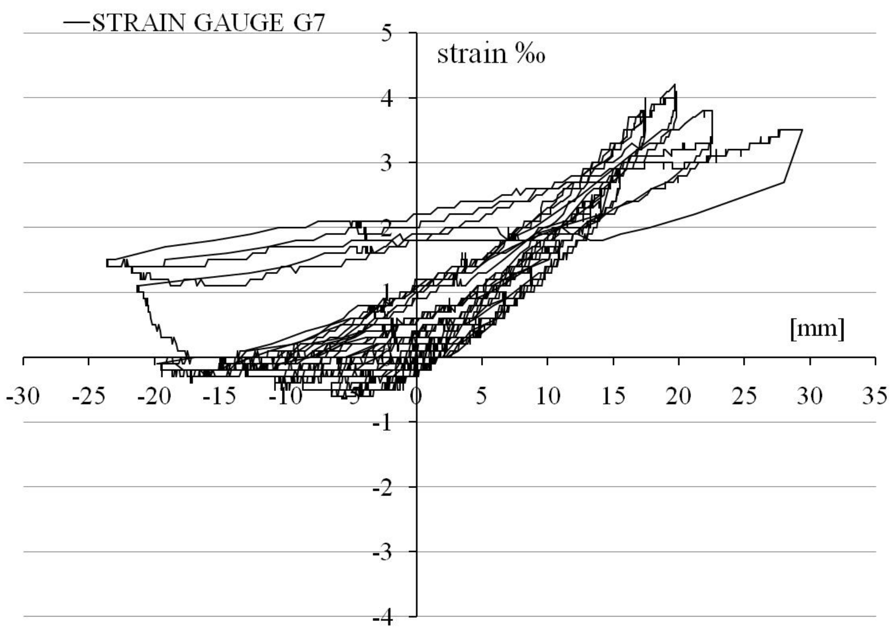


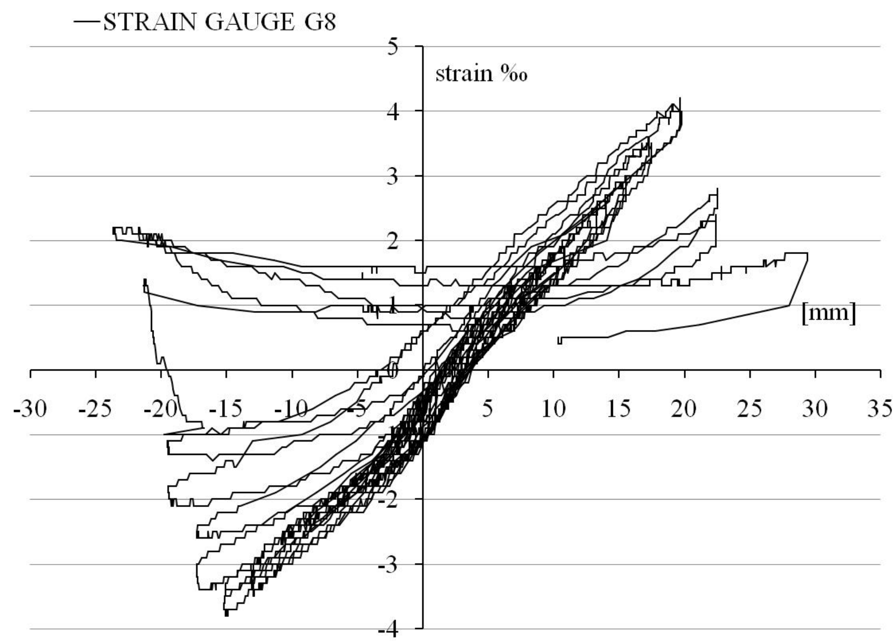


6. Results (PRCWP 9-E1/E3-R/T)

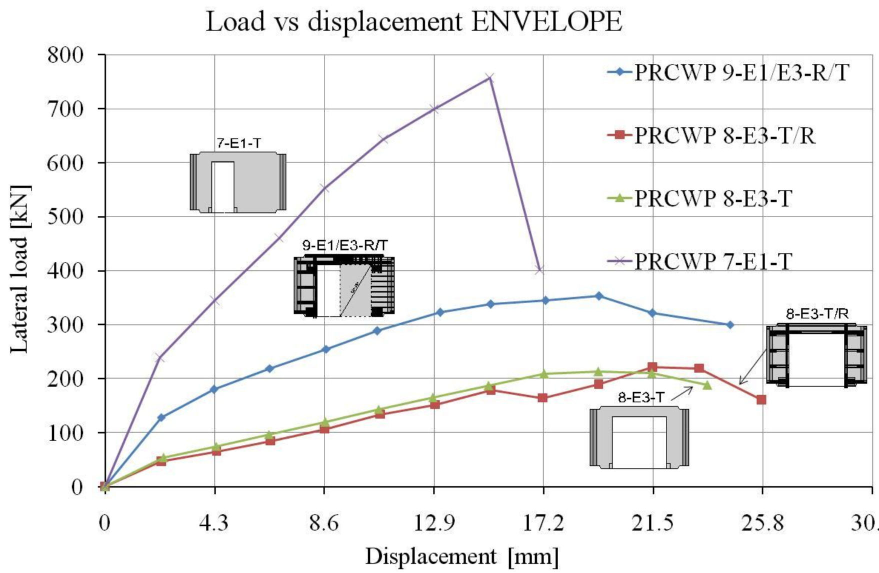


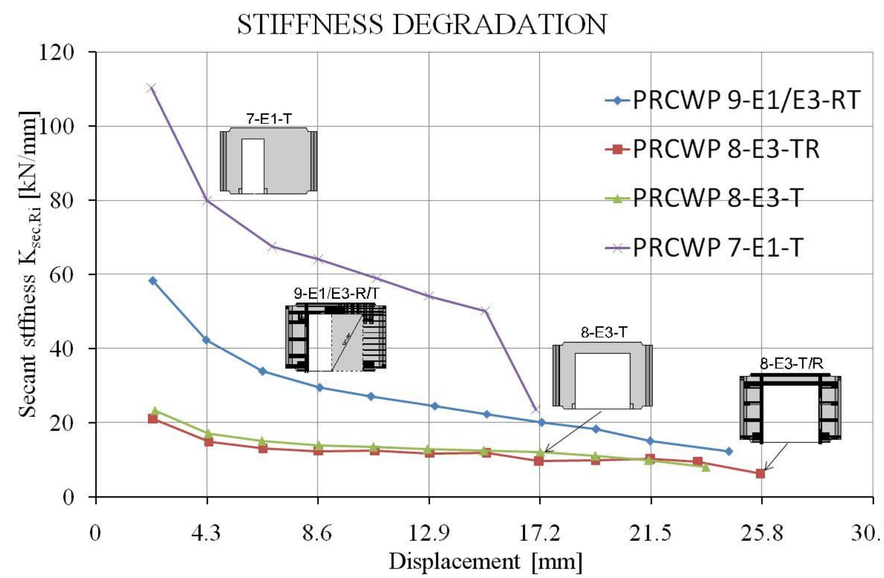


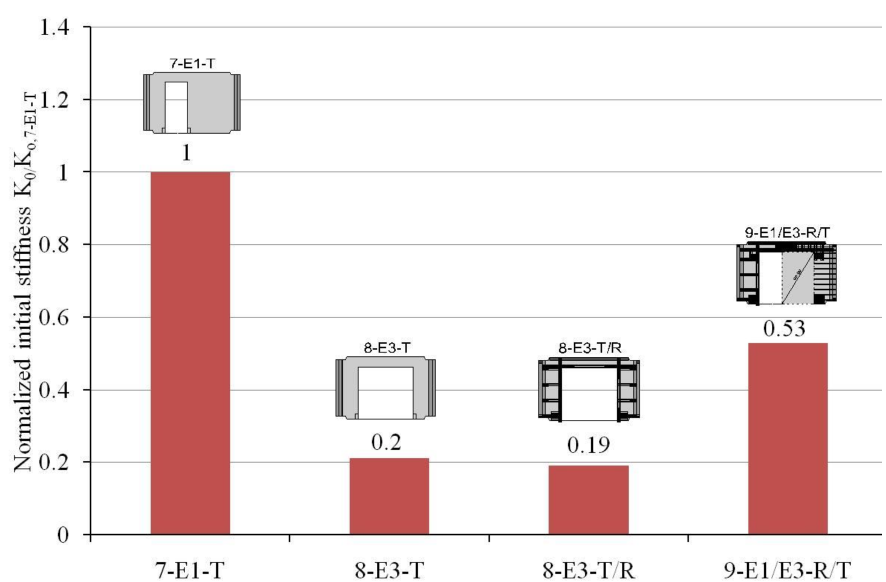


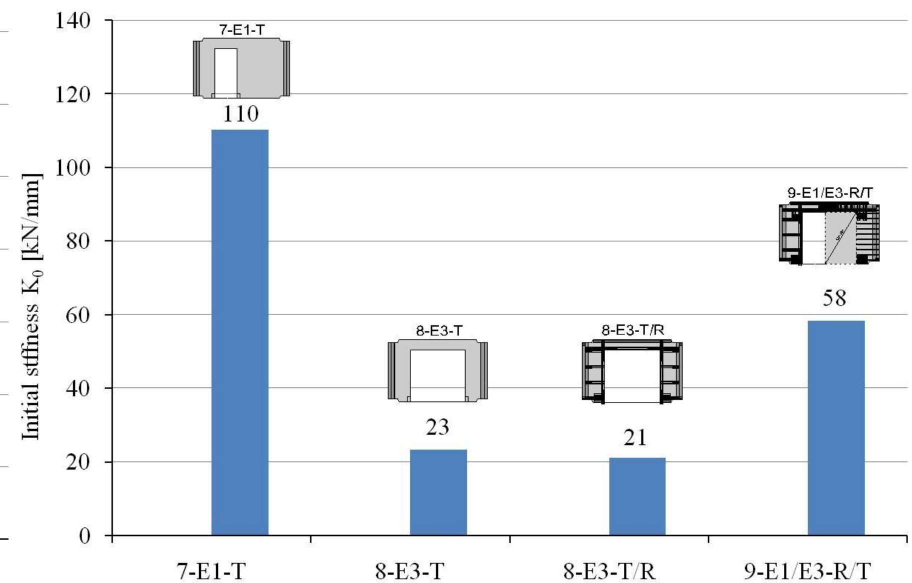


6. Results


Load vs displacement ENVELOPE
Lateral load [kN]
Displacement [mm]
PRCWP 9-E1/E3-R/T
PRCWP 8-E3-T/R
PRCWP 8-E3-T
PRCWP 7-E1-T
7-E1-T
9-E1/E3-R/T
8-E3-T/R
8-E3-T
0
100
200
300
400
500
600
700
800
0
4.3
8.6
12.9
17.2
21.5
25.8
30.


STIFFNESS DEGRADATION
Secant stiffness $K_{sec,Ri}$ [kN/mm]
Displacement [mm]
PRCWP 9-E1/E3-RT
PRCWP 8-E3-TR
PRCWP 8-E3-T
PRCWP 7-E1-T
7-E1-T
9-E1/E3-R/T
8-E3-T
8-E3-T/R
0
20
40
60
80
100
120
0
4.3
8.6
12.9
17.2
21.5
25.8
30.1


Normalized initial stiffness $K_0/K_{0,7\text{-}E1\text{-}T}$
7-E1-T
8-E3-T
8-E3-T/R
9-E1/E3-R/T
1
0.2
0.19
0.53
0
0.2
0.4
0.6
0.8
1
1.2
1.4


Initial stiffness K_0 [kN/mm]
7-E1-T
8-E3-T
8-E3-T/R
9-E1/E3-R/T
110
23
21
58
0
20
40
60
80
100
120
140

7. Conclusion

- The main objectives of the retrofit were to restore the initial load bearing capacity of the element or to obtain an increase in the value of it. In the current case the values of the load bearing capacity for the unstrengthened (7-E1-T) and strengthened specimen (7-E1-T/R) were close.

- Analysing the experimental data for specimen 8-E3, one can say that the load bearing capacity of the wall was restored. According to the strain records, the most active CFRP reinforcement was the third shear CFRP strip counted from the bottom to the top height of the panel, on the right pier, where G3 strain gauge was located. For G3, strain ranged from approximately -2.2 ‰ in compression until +3.0 ‰ in tension.

- One can remark here the significant tension-compression alternation on the diagram. During the experimental test the horizontal shear strip where G3 strain gauge was located started debonding together with concrete surface at -15.05 mm.

7. Conclusion

- The experimental test of (9-E1/E3-R/T) shows the effect of the cut-out in the wall compared to specimen (7-E1-T) and (7-E1-T/R) by a significant loss in load bearing capacity, and also the effect of initial retrofit towards (8-E3-T/R) by a significant gain in the value of the load bearing capacity, yet in the second statement we should not neglect the difference in the compressive strength of concrete, 17.5 MPa for (8-E3) and 44.5 MPa for (9-E1/E3) and the quantity of initial steel reinforcement available in the wall panel.

- Analyzing the costs we obtained a strengthening cost per square meter of 67.51 EUR/m^2 for the post-damage strengthened wall (7-E1-T/R), 57.21 EUR/m^2 for the post-damage strengthened wall (8-E3-T/R) and 94.65 EUR/m^2 for the prior-to-damage strengthened wall (9-E1/E3-R/T). The strengthening strategy using combined CFRP-EBR and NSM-CFRP was the most expensive method in this case, implying more labor and material costs.

suscos

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