Seismic Retrofitting of RC Frames with RC Infilling (SERFIN Project)

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Martin Poljansek
European Commission

Christis Z Chrysostomou
Cyprus University of Technology

Nicholas Kyriakides
Cyprus University of Technology

Toula Onoufriou
Cyprus University of Technology

European
Commission

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Martin Poljanšek, Fabio Taucer, Javier Molina Ruiz, Christis Chrysostomou, Nicholas Kyriakides, Toula Onoufriou, Panayiotis Roussis, Panagiotis Kotronis, Telemachos Panagiotakos, Antonis Kosmopoulos

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Joint
Research
Centre

European Commission
Joint Research Centre
Institute for the Protection and Security of the Citizen

Contact information
Fabio Taucer
Address: Joint Research Centre, Via Enrico Fermi 2749, TP 480, 21027 Ispra (VA), Italy
E-mail: fabio.taucer@jrc.ec.europa.eu
Tel.: +39 0332 78 5886
Fax: +39 0332 78 9049

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Work package 7

SERFIN
Seismic Retrofitting of RC Frames with RC Infilling

TA Project Final Report

Work Package Leader: Fabio Taucer, Joint Research Centre
User Group Leader: Prof. Christis Chrysostomou, Cyprus University of Technology
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Abstract

The effectiveness of seismic retrofitting of multi-storey multi-bay RC-frame buildings by converting selected bays into new walls through infilling with reinforced concrete (RC) was studied experimentally at the ELSA facility of the Joint Research Centre in Ispra (Italy). A full-scale model was tested with the pseudo-dynamic method and consisted of two four-storey (12m tall) three-bay (8.5m long) parallel frames linked through 0.15m slabs with the central bay (2.5m) infilled with a RC wall. The frames were designed and detailed for gravity loads only and are typical of similar frames built in Cyprus in the 1970's.

Different connection details and reinforcement percentages for the two infilled frames were used in order to study their effects in determining structural response. The results of the pseudo-dynamic and cyclic tests performed on the specimen with the new walls show five times higher resistance to earthquake loads when compared to typical building construction in Cyprus in the 1970's.

Keywords: Reinforced Concrete Frame, Seismic Retrofitting, Infill Walls, Experimental Analysis, Numerical Simulations, Pseudo Dynamic Tests, PsD

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Olivier Hubert

Antonio Zorzan

Giuseppe Bof

Jose Esteves

Report Contributors

Cyprus University of Technology	Christis Chrysostomou Nicholas Kyriakides Toula Onoufriou
European Commission - JRC	Martin Poljanšek Fabio Taucer Javier Molina Ruiz
University of Cyprus	Panayiotis Roussis
Ecole Centrale de Nantes	Panagiotis Kotronis
DENCO Design & Engineering Consultants	Telemachos Panagiotakos Antonis Kosmopoulos

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1 The SERFIN Project

1.1 INTRODUCTION

The construction of new walls is the most effective and economic method for retrofitting multi-storey reinforced concrete (RC) buildings, especially those with soft-storeys. Their structural and economic effectiveness increases when selected bays of an existing RC frame are fully infilled. Most of the experimental research work performed in the last decades has focused on other frequently used types of retrofitting, in particular on fibre reinforced polymers (FRP) and concrete jackets. Research on the use of RC infill walls has mainly targeted on what is feasible: testing of one- to two-storey specimens. However, data is lacking for taller full-scale specimens that reflect real life applications, due to the practical difficulties associated with the high forces needed for the tests. Regarding code provisions, Eurocode 8 – Part 3 fully covers retrofitting with FRP or concrete jackets, while it does not address the retrofitting of RC frames with the addition of new walls created by infilling selected bays. The KANEPE [9] guidelines in Greece refer to the design of such walls only in terms of forces, providing tools for calculating their deformations (at yield and failure) and stiffness only if they are integral with the bounding frame.

Experimental research on reinforced concrete frames converted into walls by infilling with RC has been carried out almost exclusively in Japan and Turkey. The experiments in Japan ([7], [8], [9], [11], [13], [17], [21]) were performed on a total of 27 1:3 to 1:4 scale single-storey one-bay RC-infilled frames with RC infill walls with a thickness of 26% to 60% (on the average 43%) of the width of the frame members. The test results were compared in most cases with monolithically cast specimens of the same geometric characteristics (in which the frame and the infill wall were cast at the same time and integrally connected). The connection of the RC infill to the bounding frame was done by means of epoxy-grouted dowels (17 specimens), or through mechanical devices, such as shear keys and dowels without epoxy (6 specimens). In four other test campaigns the thickness of a pre-existing thin wall was increased by 100% to 150% without any direct connection of the new wall with the bounding frame. The failure mode of all the specimens was in shear (including sliding at the interface). It is interesting to note that for epoxy-grouted dowels the force resistance of the infilled frame was on average 87% of the integral one, while for the mechanical connections it was 80% on average. For the increased thickness of an existing thin infill wall, the force resistance was on average 92% of the monolithic specimen, while the displacement at failure was on average 13% smaller than for the integral specimen. For the epoxy grouted dowels and for the mechanical connection the ultimate deformation was on average 55% and 115% larger than in the integral specimen, respectively. The results show that although a deformable connection gives a somewhat reduced strength with respect to the monolithic case, the ultimate deformation of the retrofitted structure is considerably increased.

Concerning the specimens tested in Turkey, those of Teymur et al. [22], Anil and Altin [2] were single storey one-bay 1:2 and 1:3 scale, with RC infill thickness 25% and 33% of the width of the frame members. Those of Altin et al. [1], Turk et al. [23], Cambay et al. [3], Sonuvar et al. [18], Kara and Altin [10] were two-storey one-bay scaled at 1:3, with infill wall thickness 33% and 40% of the width of the members of the bounding frame. The RC infill was in most cases fully connected on the perimeter with dowels, in some cases (Teymur et al. [22]) there was a gap between the infill and the columns, while in some other cases there was no connection other than simple bearing. Altin et al. [1] proposed to weld the rebars of the infill to those of the members of the frame, instead of using dowels. Only Altin et al. [1] included some monolithic specimens, but not exactly similar to the infilled ones. Finally, the specimen of Erdem et al. [6] was two-storey three-bay scaled at 1:3, with the middle bay infilled with a wall with 63% thickness of the width of the frame members. The connection was made with epoxy grouted dowels and the failure mode was predominantly flexural. In all other cases the single storey walls failed in shear, while the two storey walls failed by a combination of flexure and shear sliding at the base.

The test specimens used in the experiments described in the previous paragraphs correspond to walls with failure modes dominated by shear, with low aspect ratios not representative of multi-storey slender walls. In fact, the failure mode of multi-storey slender walls is controlled by bending and the design is governed by the formation of a plastic hinge at the base. In such a case, shear will not have a detrimental effect on displacement and energy dissipation capacity. In addition, it has been shown numerically ([5], [12]) that higher modes may increase considerably the shear forces at the upper floors of a wall after the formation of a plastic hinge at the base. This aspect has never been studied experimentally even in integral walls, because their height and number of storeys has not been large enough to allow higher mode inelastic response. Another common element of past tests is the smaller thickness of the RC infill wall relative to the width of the frame members. As a result, the weak link of the structural system is either the infill wall in diagonal compression, or its connection with the surrounding frame.

In order to start filling the gap of knowledge regarding infilling of existing RC frames with RC walls, the effectiveness of seismic retrofitting of multi-storey multi-bay RC-frame buildings by converting selected bays into new walls through infilling with RC was studied experimentally at the European Laboratory for Structural Assessment of the Joint Research Centre in Ispra (Italy). The present research was carried out within the framework of the project "Seismic Engineering Research Infrastructures for European Synergies" (SERIES), financed by the Seventh Framework Programme of the European Commission. The consortium was integrated by the Cyprus University of Technology (co-ordinator), the Ecole Central de Nantes, DENCO and the University of Cyprus. In the first part of the paper the design of the bare-frame specimen is presented and in the second part the details of the design of the RC infills are given. The results of the testing campaign are presented and conclusions are drawn.

2 Specimen Description

The reinforced concrete (RC) specimen was a four storey frame structure constructed in real scale [4]. The frame was 12.0 m high, 6.0 m wide and 8.5 m long (Figure 2.1). Along its length it had three bays but only had one bay across. The central bay in the longitudinal direction had a span of 2.5 m whereas the two exterior bays had a span of 3.0m. Storey height was 3.0m.

Firstly, the specimen was constructed on the west outside platform of ELSA without any walls and then it was transported inside. Secondly, the frame was loaded with water barrels to simulate the expected dead load on the structure and the infill walls in the central bay were poured with concrete. Water was then added to simulate the live-load.

All of the columns had the same cross section, namely 40 cm in the longitudinal and 25 cm in the transverse direction.

15 cm thick slabs were resting on beams 35 cm high and 25 cm thick. The beam's reinforcement continued into the slabs increasing its effective width.

The infill walls (Figure 2.2) in the central bay of the specimen had the same thickness of 25 cm as the columns and beams framing them. An elaborate and varying system of dowels and starter bars was used to join the walls with the frame.

The specimen was built on an 8.0 by 11.0 m foundation slab with a thickness of 40 cm, strengthened with 40 cm high and 60 cm wide beams along the direction of the columns of the RC specimen. The foundation slab had a pattern of passing holes for connecting the model with a prestressed connection to the ELSA strong floor.

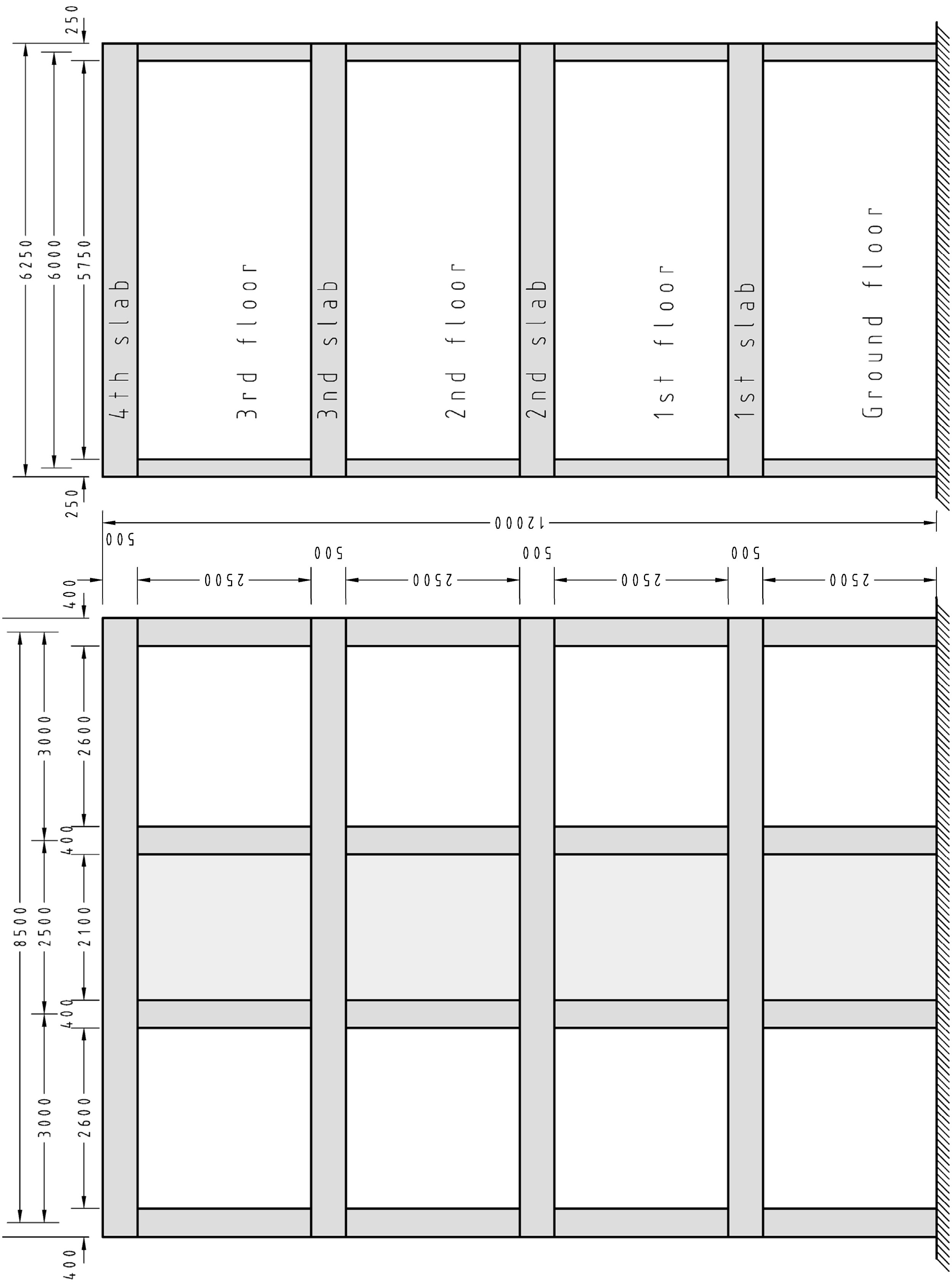

Figure 2.1 Side view of the SERFIN specimen

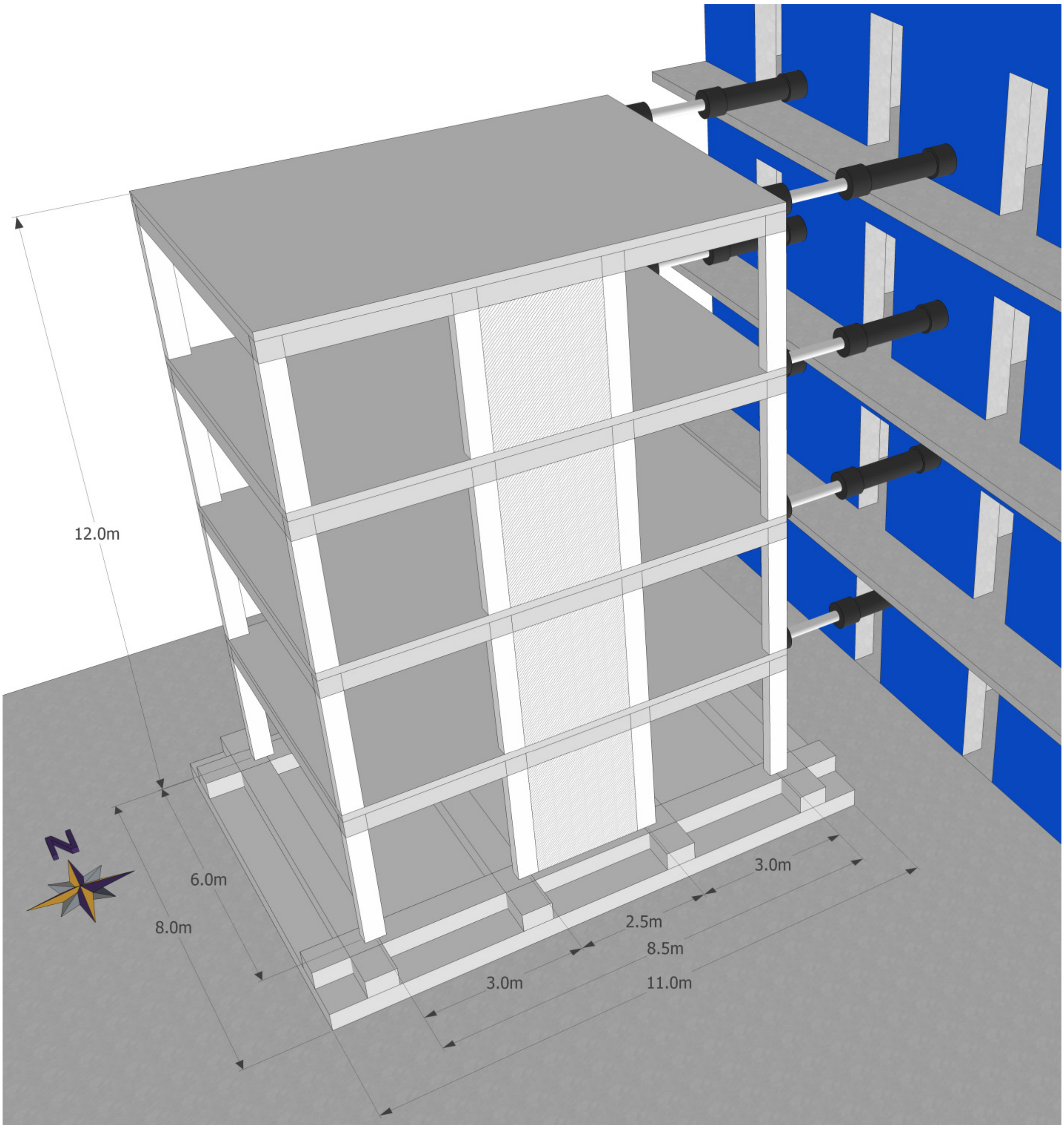

Figure 2.2 View of the SERFIN specimen with infill walls with an indication of the cardinal directions. The south frame with infill walls is visible in front.

3 Specimen Design

The SERFIN specimen (structure) represents the construction practice of the late 70's and beginning of the 80's in Cyprus. Building structures at that time were designed for gravity loads only, since there were no provisions for earthquake loading. There was no specific design standard in Cyprus and the authorities accepted standards used in other countries such as CP110 and BS8110, DIN, Greek Code, US code, etc.

The tested specimen corresponds to a four-storey prototype building frame-structure with four three-bay frames spaced at 6 m in the direction perpendicular to the seismic loading, where only the external frames are retrofitted with a wall at the central bay.
For the design of the mock-up it was decided to use the provisions of BS8110 (1983) which are very close to those of CP110 (BSI, 1972), with very minor differences. In Cyprus, the transition from CP114 (BSI, 1957), which was an allowable stress design, to BS8110, was made without going through the CP110 phase. The mock-up was designed such that all reinforcement details conformed to CP110:1972 and BS8110:1983.

The material properties used in the mock-up were constrained by the availability of materials in the Italian and European market. Concrete C20/25 was used for both the frame and the walls, with a unit weight of 25 kN/m^3 and a modulus of elasticity E=30 GPa. Deformed steel reinforcement with characteristic yield strength f_{yk} equal to 400 MPa and 450 MPa was used for all the members of the RC frame and the slab, and for the RC infill and dowels connecting the wall to the bounding frame members, respectively. The 400 MPa characteristic yield strength steel represents the one used in Cyprus construction practice in the 1970's and 80's, while the 450 MPa was the closest available in the Italian market to substitute for the 500 MPa steel that would be used today in the walls for retrofitting such a structure.

The self-weight was calculated using the unit weight of concrete specified above. The imposed dead load was 3 kN/m^2, including the load of masonry infill walls, and the live load was 1.5 kN/m^2 calculated up to the slabs edge. The above loads were combined using partial factors of safety of 1.4 for the permanent loads, and 1.6 for the live load. Material partial factors of 1.5 and 1.15 were used for concrete and steel, respectively.

4 Specimen Construction

The construction of the specimen begun in September 2010 on the west outside construction platform of the *European Laboratory for Structural Assessment* (ELSA) facility in Italy. First the structure was built on the platform, then it was transported inside where it was loaded with vertical loads (barrels filled with water). In the next phase the walls were casted and the structure was fixed to the slab of the laboratory.

4.1 FOUNDATION SLAB

The foundation slab of the structure was 11 × 8 m and was designed with four transversal and two longitudinal beams (Figure 4.3 and Figure 4.4) to provide a stiff support to the structure. Altogether there were 48.16 m^3 of concrete and 3900 kg of steel built in.

The foundation beam had 16 fixtures for lifting pistons and 72 vertical holes which were used to fix the structure to the strong floor of the laboratory once inside ELSA. Furthermore a Dywidag bar was built in the centre of each side of the slab serving as hooking points for pulling during transportation.

Pouring was done in two phases, first the 40 cm thick flat bottom part, and then the 40 cm high beams. The vertical reinforcenet of the columns was anchored in the foundation slab.

Figure 4.1 Foundation slab reinforcement

Figure 4.2 Foundation slab with column reinforcement

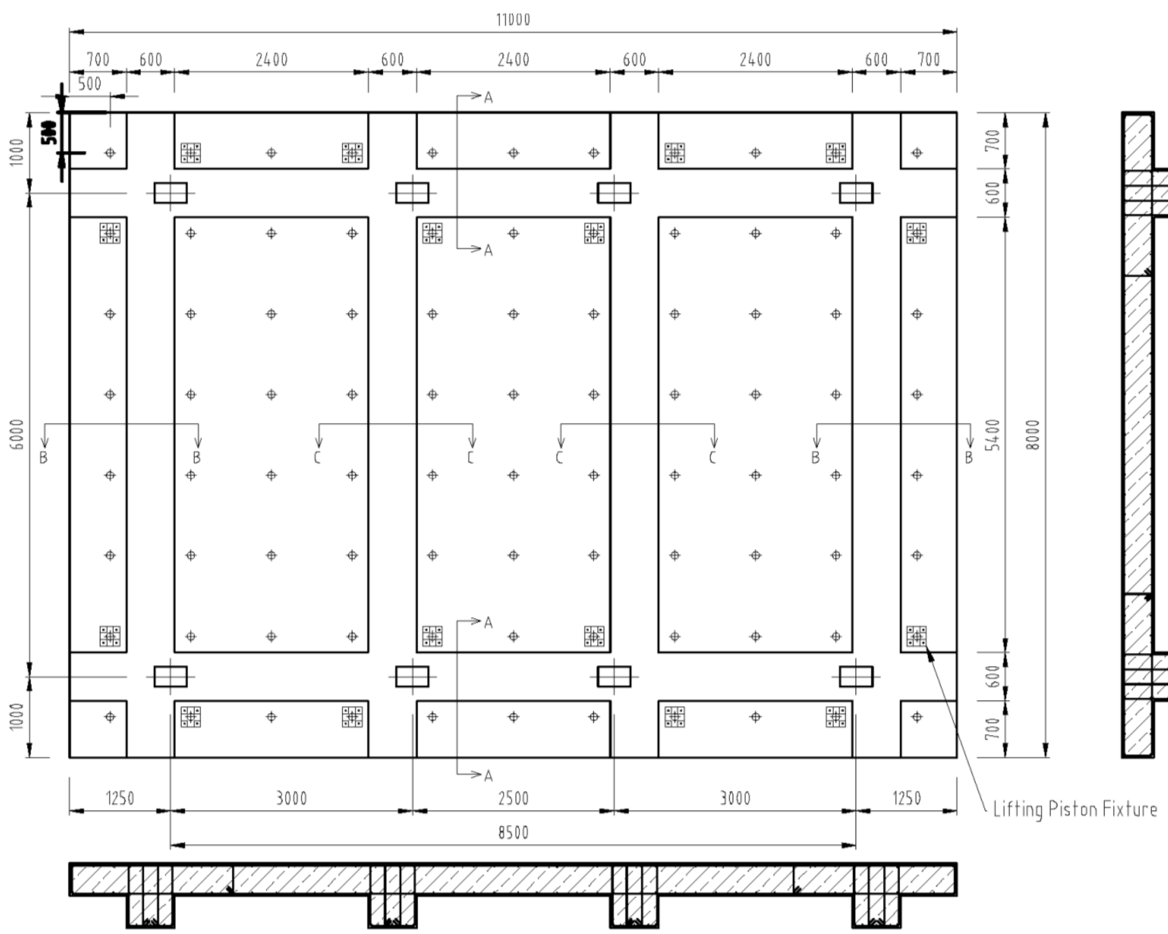

Figure 4.3 Foundation slab

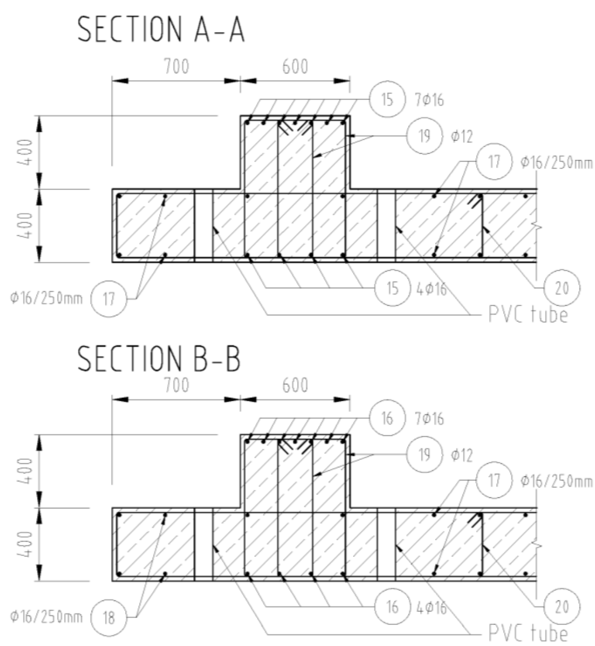

Figure 4.4 Cross section of foundation beams

4.2 SERFIN STRUCTURE

Immediately after the last pouring of the foundation slab, construction of the building structure continued. Works followed the established pattern for each storey:

1. Reinforcement of the columns.
2. Formwork of the columns.
3. Concrete pouring of the columns.
4. Reinforcement of the beams and slab.
5. Formwork of the beams and slab.
6. Concrete pouring of the beams and slab.

Every slab had 12 vertical holes to allow passing of prestressed bars used to secure the horizontal-load application beams.

The last slab was poured in late November 2010 which completed the mock-up of the bare frame. After the concrete was cured the construction workers began with the drilling of the holes for anchorages and starter bars of the wall reinforcement.

Figure 4.5 Column reinf.

Figure 4.6 Columns in the third storey

Figure 4.7 Slab reinforcement

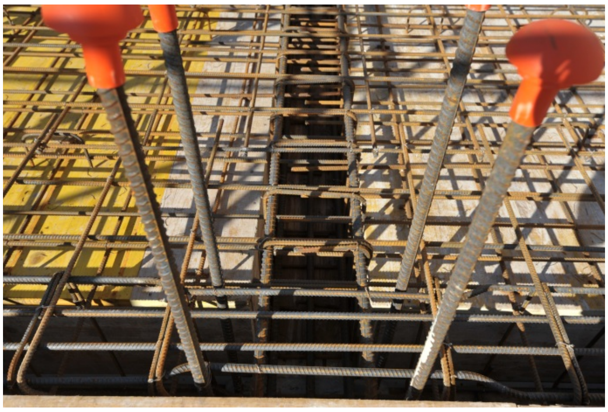

Figure 4.8 Reinforcement at a central beam

4.2.1 Columns

All columns of the structure were 400 × 250 mm and were detailed with the same longitudinal reinforcement: 4 × Φ20 ribbed bars, one in each corner (Figure 4.9). The bars were spliced right above the footing and then above each of the first three slabs with a length of 550 mm.

Φ8 stirrups were evenly spaced at 200 mm throughout the height of each column, starting at 50 mm from the top of the slab (Figure 4.5).

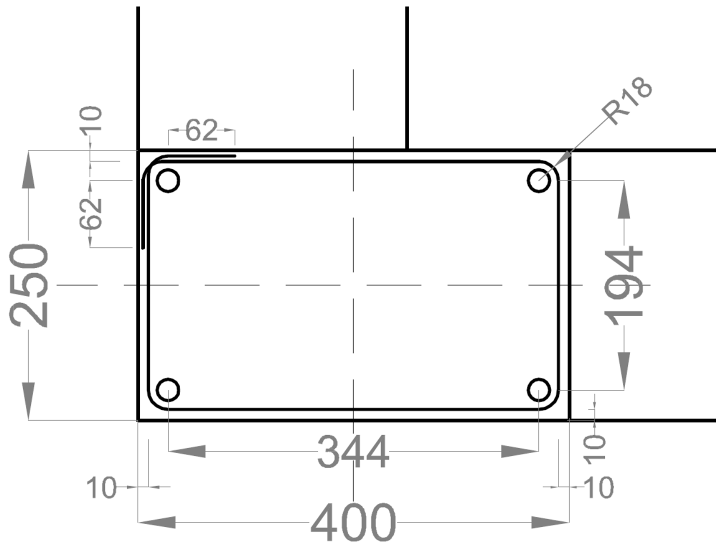

Figure 4.9 Column cross section

4.2.2 Beams

Longitudinal beams
The two longitudinal (East – West, Figure 2.2*) beams on each floor were 500 mm high and 250 mm wide, the same width as the columns supporting them (*Figure 4.10*).*

All longitudinal beams were reinforced as described below:

- Top: 4 × Φ12
- Bottom: 4 × Φ12
- Stirrups: Φ8 every 200 mm

Transversal beams

In each floor there were four transversal beams, 500 mm high and 250 mm wide, the same width as the supporting columns (Figure 4.11).

All transversal beams were reinforced as described below:

- Top: 2 × Φ20
- Bottom: 5 × Φ20
- Stirrups: Φ10 every 150 mm

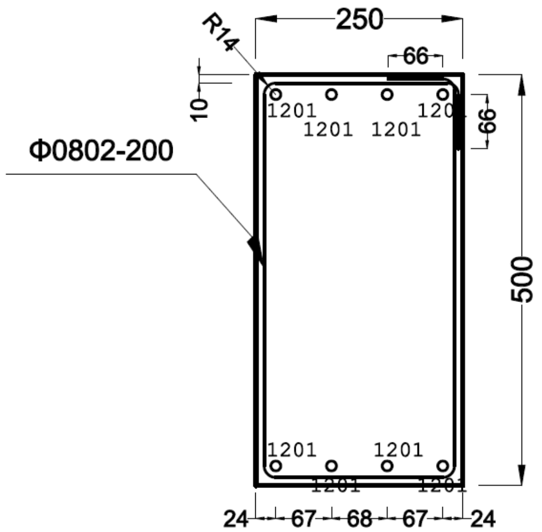

Figure 4.10 Cross section of a longitudinal beam

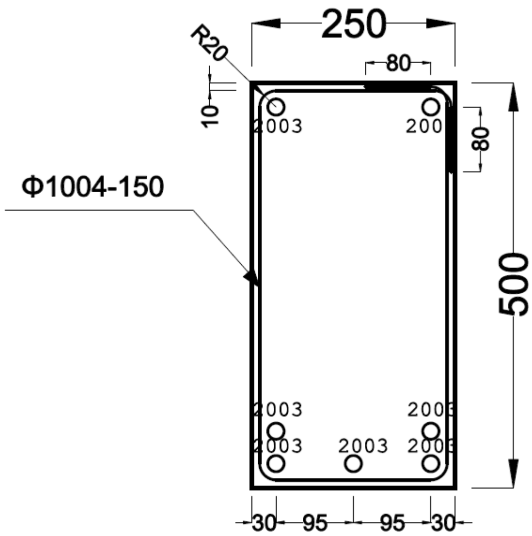

Figure 4.11 Cross section of a transverse beam

4.2.3 Slabs

The slabs were considered as elements facilitating the transfer of forces from the actuators to the two parallel frames, therefore the reinforcement was considerably increased. Although a nominal bottom reinforcement of Φ10/200 mm was required by the standard, Φ10/100 mm was specified in order to ensure adequate transfer of forces. This was necessary to avoid damage to the slabs due to high concentration of forces from the lateral load application from the actuators during the pseudo dynamic (PsD) tests. All four slabs had a thickness of 150 mm and were reinforced with steel bars at top and bottom as described below (Figure 4.12):

- Top longitudinal: Φ10 every 150 mm in the central zone and Φ10 every 200 mm at the east and west ends.

- Top transversal: Φ10 every 250 mm.
- Top around the perimeter: hooked bars Φ10 every 200 mm.
- Top corners, longitudinal direction: Φ10 every 200 mm
- Bottom longitudinal: Φ10 every 100 mm.
- Bottom longitudinal: Φ10 every 200 mm.

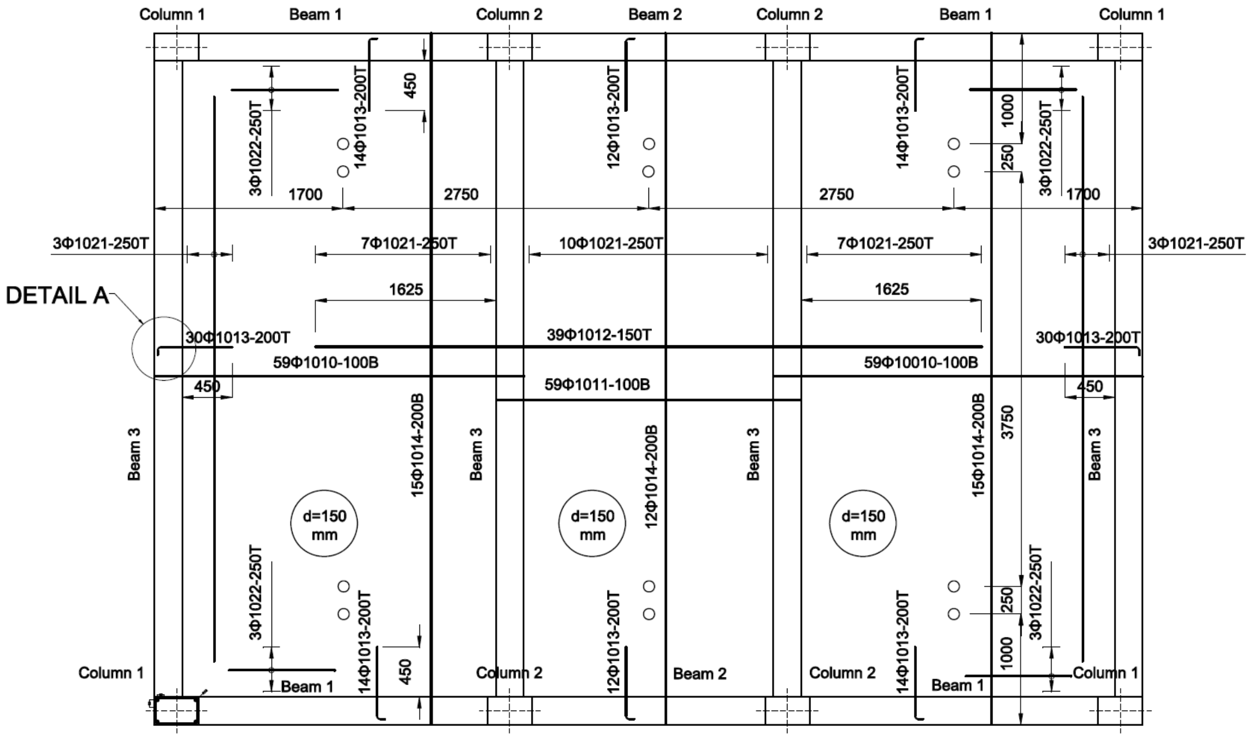

Figure 4.12 Slab reinforcement, T indicates top and B bottom reinforcement

4.2.4 Infill walls

The specimen consisted of two parallel (north and south) frames which had the central bay infilled with 250 mm thick walls as seen in Figure 2.2. The walls were reinforced with different amounts and arrangements of reinforcement, with the north wall being the stronger of the two.

Wall reinforcement and the connection details between the walls and the bounding frames are summarised in Table 1.

There were two types of connection details. In the first detail, the web bars were connected to the surrounding frame through lap splicing with the same diameter starter bars. Short dowels served to transfer shear at the interface between the wall and the frame member (Figure 4.15). This detail was used to connect the wall at the bottom beam and right column at the 1st and 2nd floors of the specimen (Figure 4.13).

In the second detail, longer bars were used to double as dowels as well as for anchorage of the web panel to the surrounding frame. The dowels are considered as lap-spliced with the nearest – smaller diameter – web bars. This detail was used to connect the wall at the top beam and left column at the 1st and 2nd floors of the specimen (Figure 4.13). In the 3rd floor of both the north and south frames only the second detail was used, while for the 4th floor only two dowels per wall interface were used to provide safety against falling of the wall out of its plane.

Table 1 Reinforcement details for the RC infill walls

Story	N Wall								S Wall							
	web bars	embedment of web starter bars, mm		Dowels					web bars	embedment of web starter bars, mm		dowels				
				Φ mm	embedment, mm							Φ mm	embedment, mm			
		in wall	in frame		bottom & east in:		top&west in:			in wall	In frame		bottom & east in:		top&west, in:	
					wall	frame	wall	frame					wall	frame	wall	frame
1	Φ12@200	600	230	Φ20	160	160	600	190	Φ10@200	500	170	Φ20	160	160	500	160
2	Φ10@200	500	170	Φ20	160	160	500	160	Φ 8@200	400	120	Φ18	145	145	400	145
3	Φ8@200			Φ18	400	145	400	145	Φ8@200			Φ16	400	130	400	130
4	Φ8@200			Φ16	400	130	400	130	Φ8@200			Φ16	400	130	400	130

The completed wall reinforcement (including web, starter bars and dowels) for the 2nd floor of the south wall is shown in Figure 4.14.

Since the lapping of the column reinforcement could take only compression, a lap splice failure would have taken place during the test, which could be detrimental to the whole experiment. Therefore, in order to safeguard against this type of failure and allow the experiment to be performed without any premature failure, it was decided to reinforce the edges of the wall at the 1st floor with three-sided CFRP jacket with a height of 0.60 m from the base of the column (Figure 4.16).

Figure 4.13 Dowels and starter bars

Figure 4.14 Dowels, starter bars and web reinf.

Figure 4.15 Installation of dowels and starter bars

Figure 4.16 CFRP reinforcement jacket of the ground columns in the north frame

Figure 17 Construction phases of the SERFIN structure

5 *Loading*

5.1 VERTICAL LOADING

It was assumed that each floor should be loaded with 3 kN/m^2 of dead load and 30% of 1.5 kN/m^2 live load. Thus, (1.0 × 3.0 kN/m^2 + 0.3 × 1.5 kN/m^2) × 6.25 m × 8.90 m = 192 kN was applied per storey. 135.4kN was applied with 15 water barrels (Figure 5.1) and the rest was the self-weight of the actuator attachment beams. The structure was loaded with the dead load before casting of the infill walls, with the live load added after the casting, to simulate real situation where an existing building is retrofitted.

Because of the slow nature of the PsD method, the water did not show any dynamic effects during the tests.

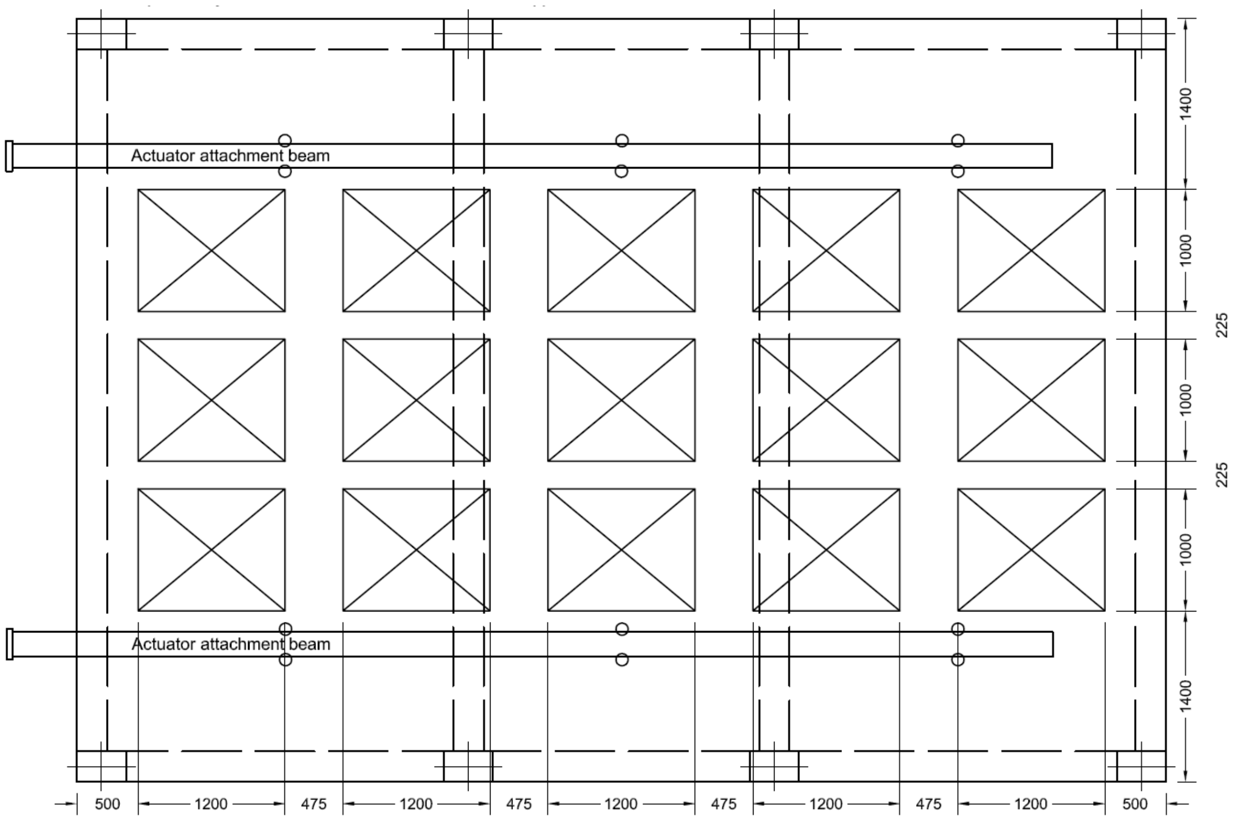

Figure 5.1 Water barrels arrangement


Figure 5.2 Filling the water barrels


Figure 5.3 Vertically loaded structure; infill walls pouring

5.2 HORIZONTAL LOADING

In each storey a pair of servo hydraulic actuators applied horizontal loads as derived from the PsD test method. To connect an actuator to the structure a system of steel beams was installed. One beam was passing above and the other under the slab. The beams were then clamped together with prestressed Dywidag bars. On the actuator side they were welded together to a sheen plate to which the actuators were bolted (Figure 5.5). Spacers were designed to allow the bottom beam passing without leaning against the beams of the structure (Figure 5.4). All actuator generated load was thus transmitted to the structure by friction minimizing stress concentrations. This ensured smooth transmission of forces also when the direction of loading was changed.

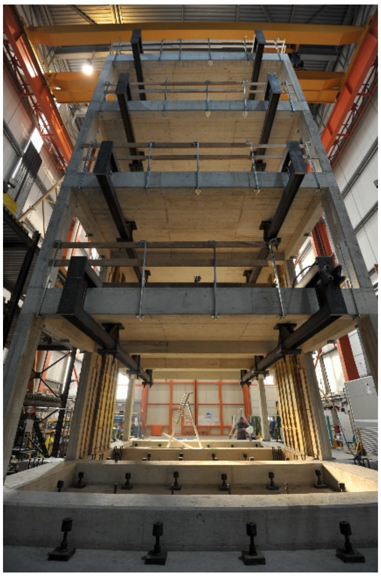

Figure 5.4 Actuator attachment beams

Figure 5.5 Detail of an actuator attachment beam

6 Instrumentation

The SERFIN test structure was instrumented with 108 potentiometric displacement transducers (Gefran), 22 inclinometers, 8 Heidenhain linear encoders and 8 load cells built in the actuators. The arrangement of instrumented zones was symmetrical for the two frames. The north ground floor wall and its bounding columns and beams was monitored with a pair of high resolution, water cooled Long Wording ForPco (PCO) Edge digital cameras.

Displacement transducers were installed to measure local displacements in critical areas. In particular, transducers were placed to monitor: slip and crack opening between all walls and their bounding beams and columns, the displacements between the ground floor walls and the foundation beams, and the shear deformations of the two ground floor walls. Displacement transducers were also installed to measure the vertical elongation of the bounding columns at all stories.

Inclinometers were used to measure the rotation of beams and columns at the first floor. They were placed at the centre joints and on beams and columns 30 cm away from the joints. Inclinometers were also placed at selected columns 30 cm above the foundation beam.

Heidenhain linear encoders were installed on two reference frames to measure the horizontal displacement of the two frames at each of the four floors in the direction of testing. They served as reference displacement instruments for control in all tests.

The part of the structure in the field of view of the two digital cameras was painted with a random speckled pattern. During post processing of the images this allows to recreate a 3D displacement field of the area covered by the pattern and to identify the occurrence of cracks and follow their progression. This system allows an accurate measurement of displacement and crack openings, which will be an invaluable resource of data for the study of the local behaviour of the specimen.

In some parts of the tests, data acquisition problems of some channels occurred. In such cases flat zero response of the instruments was inserted in post processing of the results.

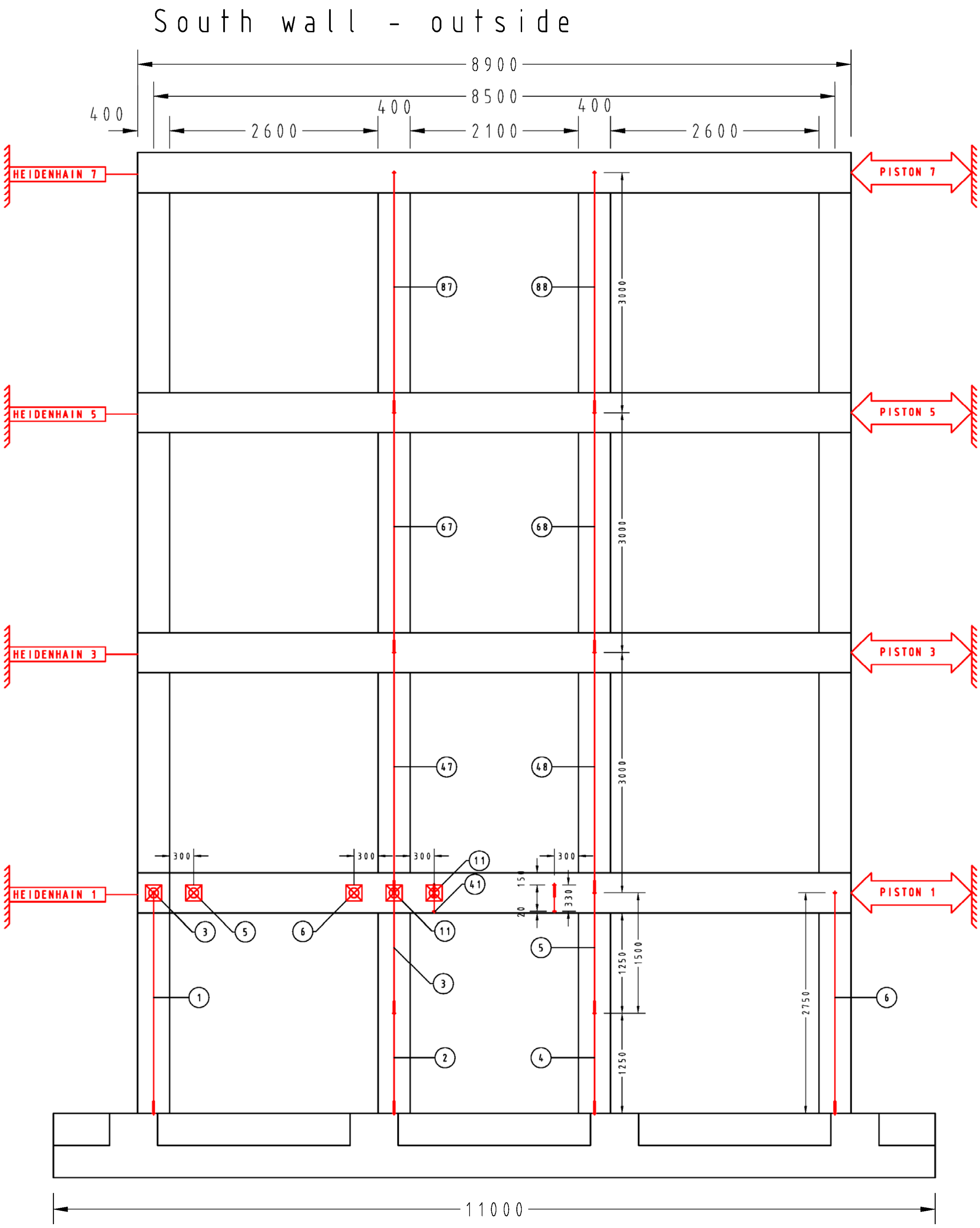

Figure 6.1 Instrumentation; South frame - outside

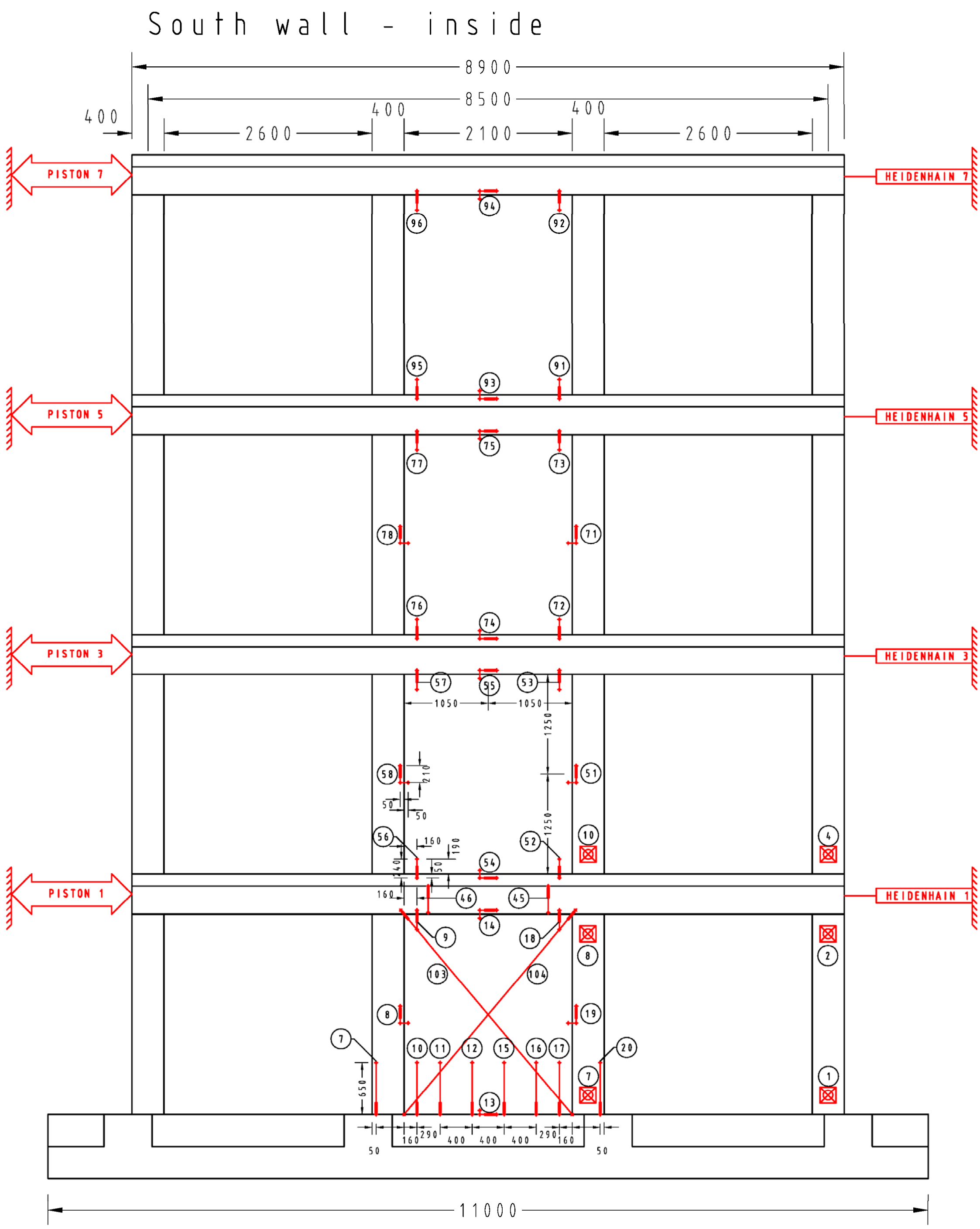

Figure 6.2 Instrumentation; South frame – inside

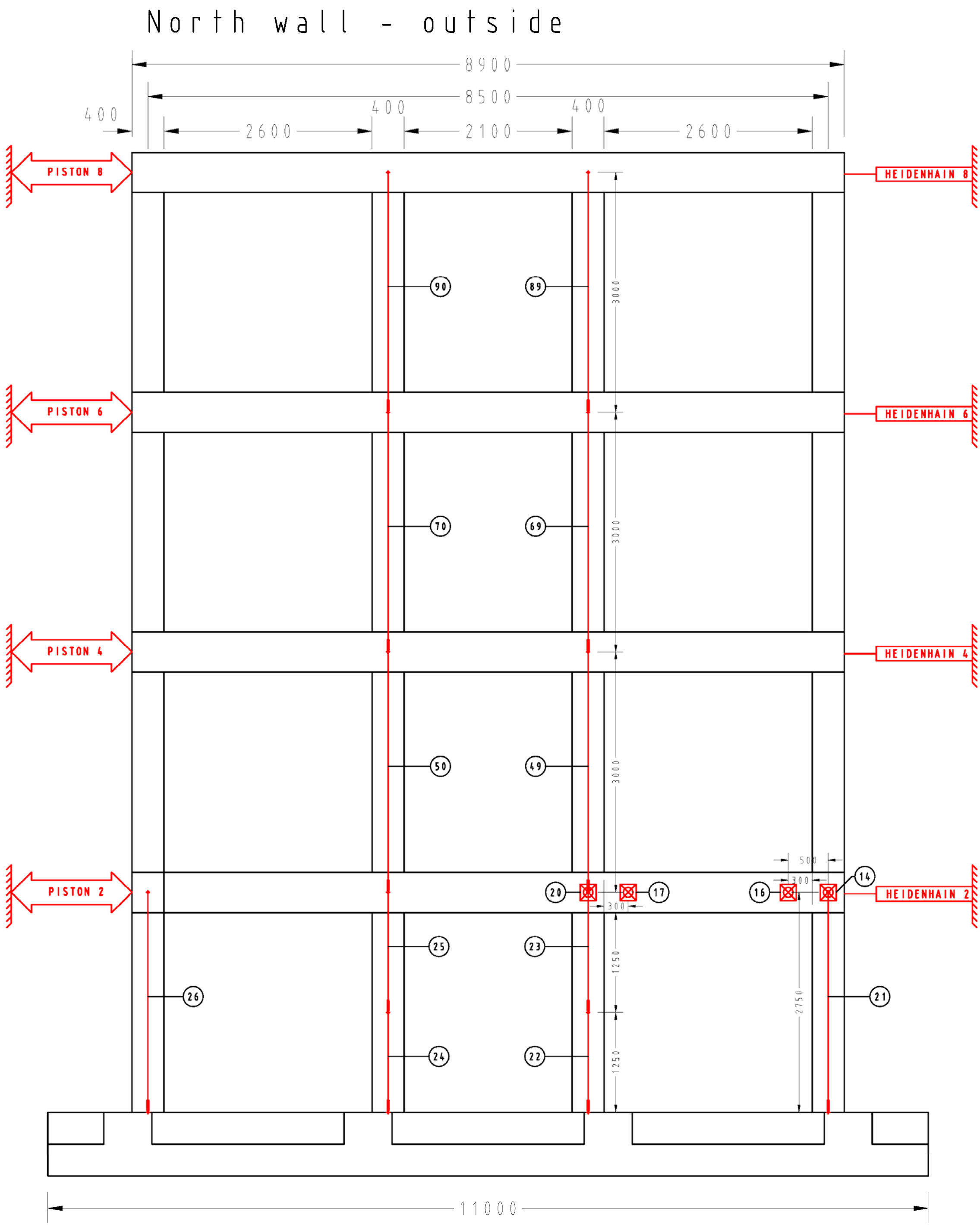

Figure 6.3 Instrumentation; North frame - outside

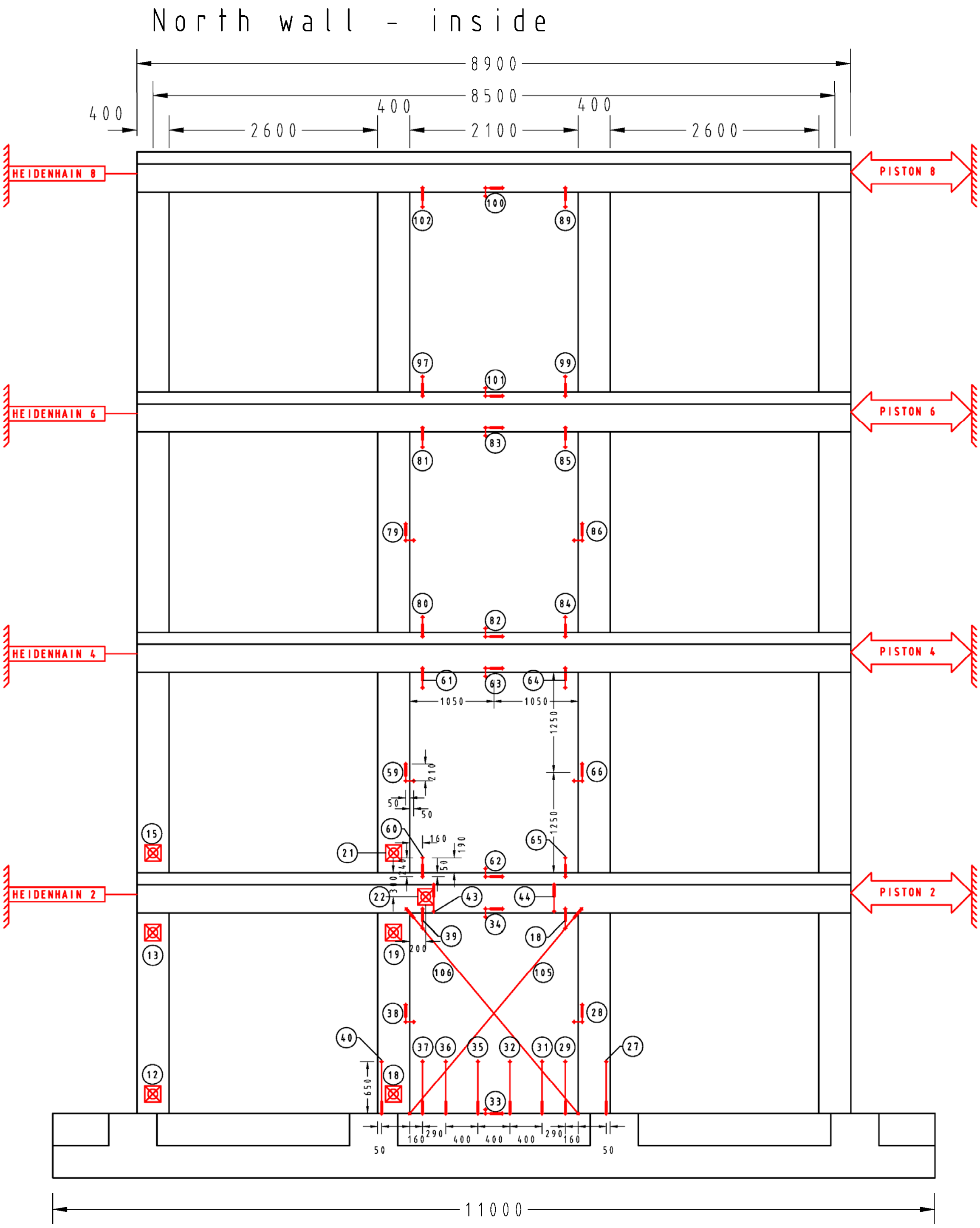

Figure 6.4 Instrumentation; North frame - inside

Figure 6.5 Ground floor north wall instrumentation

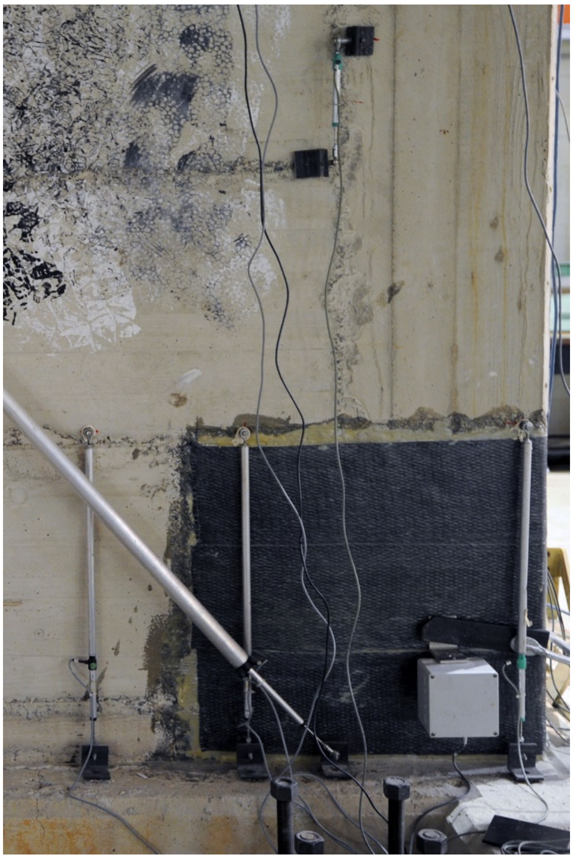

Figure 6.6 Detail of instrumentation at the bottom of a ground column

Figure 6.7 Instrumentation on the outside face of the central bay, north frame

7 Experimental Method and Set-up

7.1 PSEUDO DYNAMIC METHOD

In a pseudo dynamic (PsD) test on-line computer numerical models are combined with actual measurements of the properties of a structure. To simulate the response of a structure under seismic loading the computer running the PsD simulation takes an accelerogram as an input. For the test campaign, the first 15 seconds of the transverse component of the Herzeg Novi (Montenegro 1979) accelerogram (Figure 7.1) was adapted to the EC8 response spectrum of a Type B soil, digitized in 0.005 s steps and scaled to a target peak ground acceleration.

In a PsD test it is assumed that the response of a structure can be determined by a discrete model with a limited number of degrees of freedom (DoF). In this test campaign four DoFs were selected: the horizontal displacements of each storey with the assumption that all the mass is concentrated at the selected DoFs (i.e., the floor slabs). The equations of motion for such an idealized system are second order differential equations which can be expressed in matrix form:

$$M \cdot a(t) + C \cdot v(t) + r(t) = f(t)$$

Where M is the mass matrix, C is the viscous damping matrix, $r(t)$ is the internal (restoring) force vector and $f(t)$ is the external force vector applied on the structure. Horizontal displacements of the controlled DoFs were solved for a prototype time step of $0.005/2000=2.5\times10^{-6}$ s using the explicit Newmark time integration method. To guarantee optimal control of structure response the equation of motion was solved for the north frame and the resulting displacements were applied to both the north and south frame. Displacements were then applied by horizontal actuators at each storey at a laboratory time step of 0.002s corresponding to the sampling rate of the controllers. The forces measured by the load cells in the actuators, following the application of the controlled displacements, represent the restoring forces that are fed back to the computer and that are used in the next time step of the calculation. Restoring forces are thus obtained from the specimen’s response and reflect its state of damage.

Since the inertial and viscous damping forces are modelled in the computer the test does not have to run in real time scale. The hysteretic damping is automatically accounted for through inelastic deformation and damage progression of the test structure; consequently no viscous damping matrix was used [14]. During the PsD test campaign the equation of motion was solved for restoring forces coming from the north frame only (calculated from static equilibrium of the load cell force measurements at each floor) and multiplied by a factor of two (the south frame is considered equal to the north frame in the numerical model). Equal displacements were applied to the two frames, in order to maintain zero rotation along the horizontal plane of the floor. The PsD test method used for the test campaign was continuous, which reduces problems of material relaxation and avoids load over-shoot [16], [15].

The mass used in the equations of motion of the PsD test corresponded to the total mass of the prototype, equal to 156 tons per floor, under the assumption that the two inner frames of the prototype building provide a negligible stiffness contribution in the direction of the lateral, seismic forces.

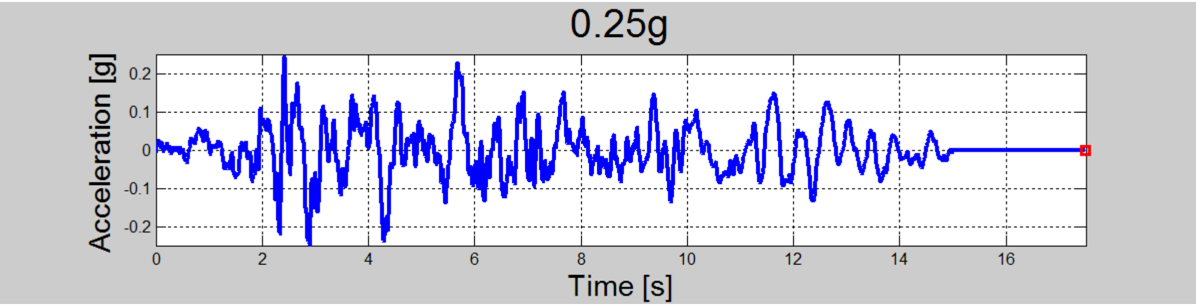

Figure 7.1 Herzeg Novi (Montenegro 1979) accelerogram scaled to 0.25g

7.2 EXPERIMENTAL SET-UP

In the pseudo-dynamic (PsD) tests eight actuators (two 1000kN actuators in the two top floors of the specimen and two 500kN actuators at the lower two floors) imposed the controlled displacement at each floor of the structure (Figure 7.2). At each floor there were two actuators symmetrically positioned distanced at 4.05m between their axes. The actuators were braced against the reaction wall and at the specimen end pinned to a loading U interface consisting of two 8.5 m HEB 200 longitudinal beams welded to a transverse element at the actuator connection. One of the beams rested on the top surface of the slab (Figure 5.5) while the second was positioned beneath the transverse beams. The steel beams were clamped at three points along their length with prestressed bars passing through the slab. This configuration allowed a near to uniform distribution of load from the actuators to the floor slab.

Figure 7.2 Sefin specimen ready for testing

The displacement control of each floor was done with linear encoders (Heidenhain) that provided reference floor displacement data.

The displacement control typically used for PsD tests encountered some difficulties of stability due to the high stiffness of the specimen (owing to the presence of the RC infill walls) that showed an eigenfrequency of approximately 30 Hz at the 4^{th} mode of vibration of the specimen. The test was made stable by using low values of the proportional parameter of the control. A numerical model of the control system was also used to improve the tuning of the proportional integral derivative (PID) control loop [15]. In order to guarantee stable behaviour in the PsD response at the highest mode of the specimen and minimum error of the modes present in the response, the test was conducted 800 times slower with respect to the real duration of the accelerogram used as input.

8 Test campaign

The test campaign on the SERFIN specimen started with a series of small tests with the input accelerogram scaled to 0.02g and the maximum of applied forces limited to ±200 kN. The limitations were introduced to keep the specimen undamaged but still acquire representative results necessary to check all equipment, instrumentation and set the parameters of the test controllers.

Within the testing campaign two PsD tests and one cyclic test were run. The accelerogram was scaled to the maximum peak ground acceleration of 0.10g and 0.25g for the first and second PsD tests, respectively. For the final cyclic test, a history of displacements was imposed at the fourth floor, while maintaining a triangular distribution of loads along the height of the north frame and zero rotation at each of the four floors. The aim of this test was to explore the final capacity of the specimen up to a 20% drop of peak strength of the structure.

The results from the test campaign are provided for the following sequence of tests:

- 0.10g PsD Test
- 0.25g PsD Test
- Final Cyclic Test

To simplify description of failures the following numbering scheme is shown in Figure 8.1.

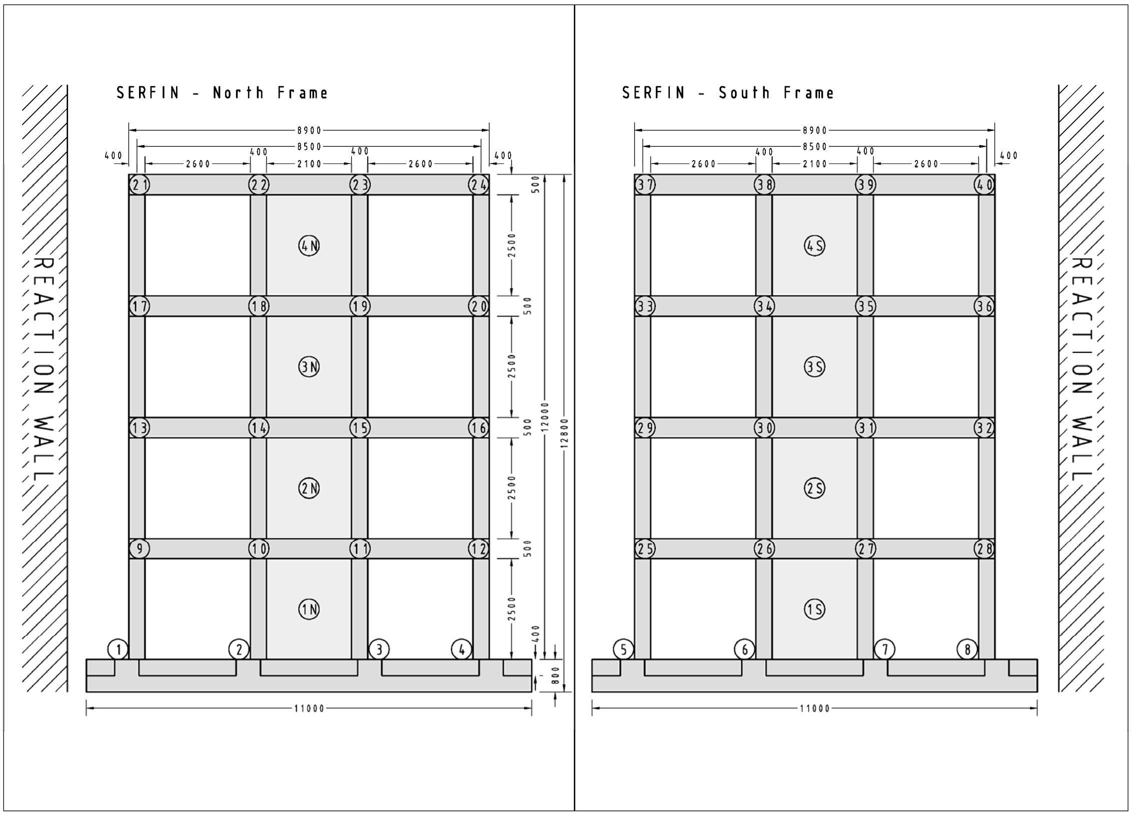

Figure 8.1 Numbering scheme of positions in the north and south frame

8.1 0.10G TEST

The 0.10g test was designed to induce minimum damage on the structure. Indeed, after a visual inspection of the specimen, no visible cracks on the columns or walls could be noted. Some hairline cracks that appeared on the surface of the wall at maximum displacement closed down at the end of the experiment. The recorded maximum top storey displacement was equal to 24 mm (towards the reaction wall) and -25 mm in the opposite direction (Figure 10.1).

It should be noted that the variation of the displacements at the first and second stories is shown only up to about 11 seconds, since a problem was encountered with the data acquisition system. The variation of the base shear with the top displacement in each of the frames is shown in Figure 10.9. As it can be observed from the figure, there is very little difference between the two frames. The maximum positive shears were 645kN and 574kN and the maximum negative shears were -634kN and -625kN, for the south and north frames, respectively. Based on the results of the test and the observed damage, it can be considered that both walls reached their cracking moment.

Local measurements (Figure 10.11 to Figure 10.20) were also affected by acquisition hardware problems and most local data was lost after about 7 seconds of the test.

8.2 0.25G TEST

The 0.25g test was designed to study the performance of the specimen at its ultimate capacity. The maximum top storey displacements were 109 mm and -93 mm (Figure 10.21). Some differences were observed in the base shear between the two frames. As it can be observed in Figure 10.29, the maximum base shear in the positive direction (towards the reaction wall) was 1074kN for the south frame and 1036kN for the north frame, while a larger difference was observed for the negative base shear: -843kN for the south frame and -1011kN for the north frame (at the same displacement time step), providing an indication that the south frame had suffered larger levels of damage than the north frame. This was confirmed by a crack that opened at the ground beam of the foundation at the base of the wall and by a lap-splice failure that appeared in the outer column at the east side of the south frame. CFRP jackets on the bounding columns of the wall prevented a similar failure, thus allowing completion of the experiment.

Examining Figure 10.27 and Figure 10.28 it can be observed that there is a steady decrease of the interstorey shear from about 1000kN at the 1st storey to about 400 kN at the top storey. What it can be also seen is that while the interstorey drift at the 1st storey is about 20 mm, the interstorey drift for the upper three floors is on the order of 30 mm, which shows the influence of the larger stiffness of the RC infill wall connection at the base. It can be also observed that the hysteresis loops are stable and provide for some energy dissipation.

The first failure that could be observed visually was at the bottom of the wall in the south frame. The crack opened under the CFRP jacket at the ground floor as it can be seen in Figure 8.2 and Figure 8.4. In the northern frame such a failure was not observed, but more will be known when the results of the optical metrology become available.

Following the failure in the south frame wall also the column in the column closest to the reaction wall suffered failure in the lap splice zone (Figure 8.3). Throughout the test this column suffered the most severe damage but continued to carry compression load, whereas it was visible that the joint failed in tension. The column on the opposite side (position 5) did not suffer damage, apart from some cracks in the lap splice zone.

Cracks developed in west columns of both the north and south frames at the first and second storey (Figure 8.5 and Figure 8.7). The cracks, their position and concrete spalling in the latter phase suggest lap splice failure in tension. West columns in the ground floor remained without visual damage.

Hairline cracks developed also in the ground floor of the south wall (Figure 8.6) but no major damage was observed in the wall or the connection with the surrounding frame. However a horizontal crack did appear in the foundation beam.

Concerning the general behaviour of the specimen during the two PsD tests, its performance was in accordance with the damage expected for the retrofit design corresponding to a life-safety limit-state for the 0.25g earthquake (475 years return period). There were no visible diagonal cracks on the walls, confirming that the wall responded in flexure. In nearly all of the corner columns and at all floors, a horizontal crack appeared at a height of 0.55 m, corresponding to the limit of the lap-splice; in some cases spalling of the concrete cover was observed. Some vertical cracks appeared in the beams close to the beam - column interface, but no severe damage was observed, despite the fact that there were no ductile connections in the structure. In general, the stronger north frame had an overall better behaviour compared to the south frame; nevertheless the differences between the two frames were minor.

Figure 8.2 Position 6: Crack opens at the bottom of the column - wall

Figure 8.3 Position 8: Failure of the column at the bottom

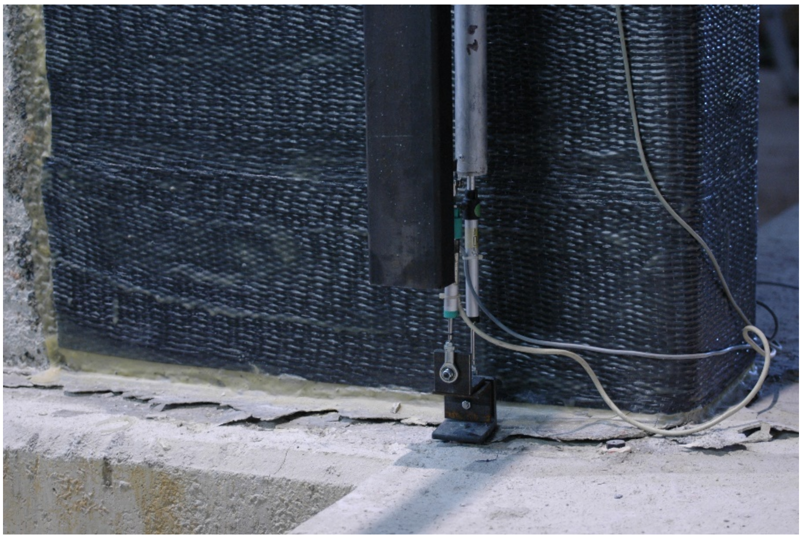

Figure 8.4 Position 7: Crack opens at the bottom of the column - wall

Figure 8.5 Position 12: Failure of the laps splice joint, N frame

Figure 8.6 Position 1S: Hairline cracks develop in the wall

Figure 8.7 Position 29: Failure of the lap splice joint, S frame

8.3 FINAL CYCLIC TEST

During the final cyclic test a displacement history was imposed at the top storey (92, -92, 89, -125, 37, 0 mm). The objective of the test was to obtain a 20% reduction of the peak strength of the specimen, so as to complete the global force-displacement envelope of the specimen. The base shear versus the top storey displacement of the cyclic test is shown in Figure 10.49. As it can be observed in the first cycle the structure was able to reach 92 mm in both directions. In the second cycle the objective was to study the response of the specimen at 125 mm. However, although in the negative direction a displacement of 125 mm could be reached, the test could not go beyond 89 mm in the positive direction due to saturation of the second floor actuator in the south frame. This happened because in an attempt to keep the same top storey displacement of the two frames while maintaining a lower base shear in the weaker south frame, the required force in the second storey of the south frame in the direction opposite to the displacement of the structure exceeded the actuator capacity. The force-displacement envelope in the negative direction shows that the strength of the south frame dropped from -838kN at -110mm to -553kN at a displacement of -125mm. This amounts to a drop in strength of 34%, larger than the target of 20%. After that, the top displacement was reduced to 37 mm in the positive direction and from there to zero.

In the last test the horizontal crack opened further at the bottom of the north and south wall (Figure 8.9, Figure 8.10 and Figure 8.12). The crack ran under the CFRP jacket which kept other parts of the columns undamaged. Cracks also developed in the foundation beams (Figure 8.9) under the walls, which compromised local displacement measurements of the transducers fixed at the foundation

Both ground floor columns closest to the reaction wall (Figure 8.11 and Figure 8.15) were damaged in the lap splice zone, both columns failing in tension but with some remaining vertical load carrying capacity. As in the 0.25g test, damage to the southern frame was much more significant.

All beams next to the walls at all storeys were damaged in the cyclic test, typically as seen in Figure 8.8. The damage indicates yielding of the longitudinal reinforcement. In spite of the joints not having been designed for ductility, they retained most of their bending capacity, as the level of damage was minor.
The remaining beam-column joints suffered moderate damage, which was most pronounced in the western part of the two frames. The cracks developed at the bottom of columns and propagated into the beams (Figure 8.14).

In the ground floor cracks between the infill wall and the surrounding frame were noticed (Figure 8.13), indicating slip between these two elements. However, local measurements show that the horizontal slip (between wall and beam) remained under 0.8mm (Figure 10.56), whereas the vertical slip (between wall and column) remained under 0.4mm (Figure 10.57).

Figure 8.8 Position 30: Beam next to the wall is cracked

Figure 8.9 Position 7: Horizontal crack developed also in the foundation beam

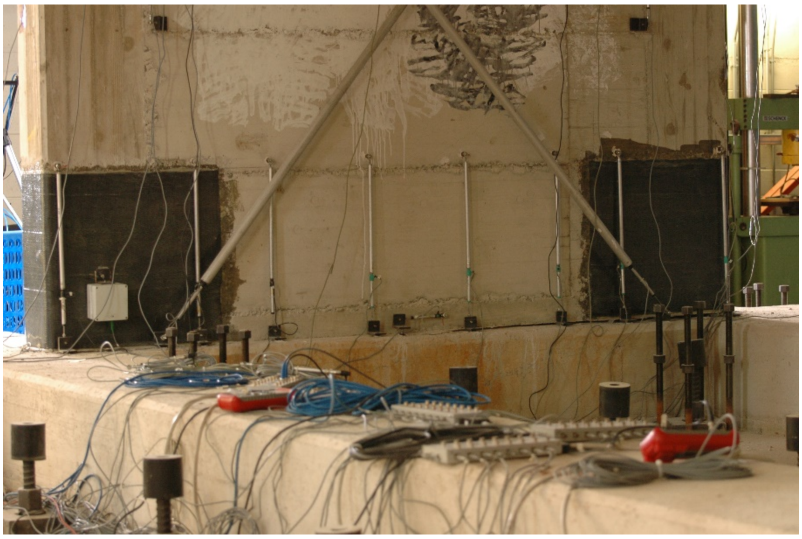

Figure 8.10 Position 1N: A crack opened also in the northern wall at the bottom

Figure 8.11 Position 1: Damage to the north wall column

Figure 8.12 Position 2: Crack opened under the CFRP jacket

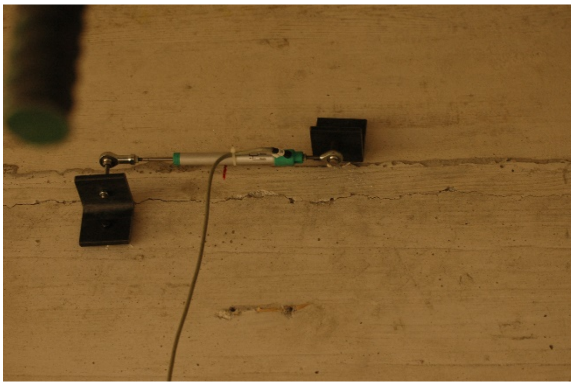

Figure 8.13 Position 1S: Slip crack at the top of the southern wall in the ground floor

Figure 8.14 Position 29: Cracks propagated into the cross beam

Figure 8.15 Position 8: Column is severely damaged

9 Conclusions

The effectiveness of seismic retrofitting of multi-storey multi-bay RC-frame buildings by converting selected bays into new walls through infilling with RC was studied experimentally on a full scale specimen at the ELSA facility of the Joint Research Centre at Ispra. The main parameters of the mock-up were the connection between the RC infill and the surrounding RC frame and the percentage of reinforcement in the RC infill. The effect of these parameters was studied during the experiment, by using different connection details and reinforcement percentages for the two infilled frames. The main findings of the test campaign are:

- The structure managed to sustain an earthquake of 0.25g without significant damage.
- Some column lap-splices failed with concrete spalling, but the structure continued to carry load.
- The three-sided CFRPs protected the wall bounding columns at the ground floor and prevented lap-splice failure.
- The “weak” south frame behaved equally well as the “strong” north frame.
- The slip-displacement at the horizontal interfaces of the ground-floor walls were on the order of 0.8mm, which is very close to full engagement of the starter bars, but not of the dowels.
- The slip-displacements between the wall and the bounding columns of the ground-floor was on the order of 0.4mm.
- The two connection arrangements used performed satisfactorily, but no solid conclusions can yet be drawn regarding the advantages of the one over the other.
- Some vertical cracks appeared at the connection of the beams to both the exterior and the wall columns.
- Horizontal cracks appeared at the foundation beam under the walls, which was the main cause for the loss of strength of the south frame.

It was demonstrated that this is a viable method for retrofitting and it can be used to strengthen existing ductility and strength deficient structures. The recorded global and local behaviour of the structure provides data for the development of numerical models, thus facilitating the proposal of design guidelines for such a retrofitting method.

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10 Annex 1: Test Results Diagrams

0.10G TEST

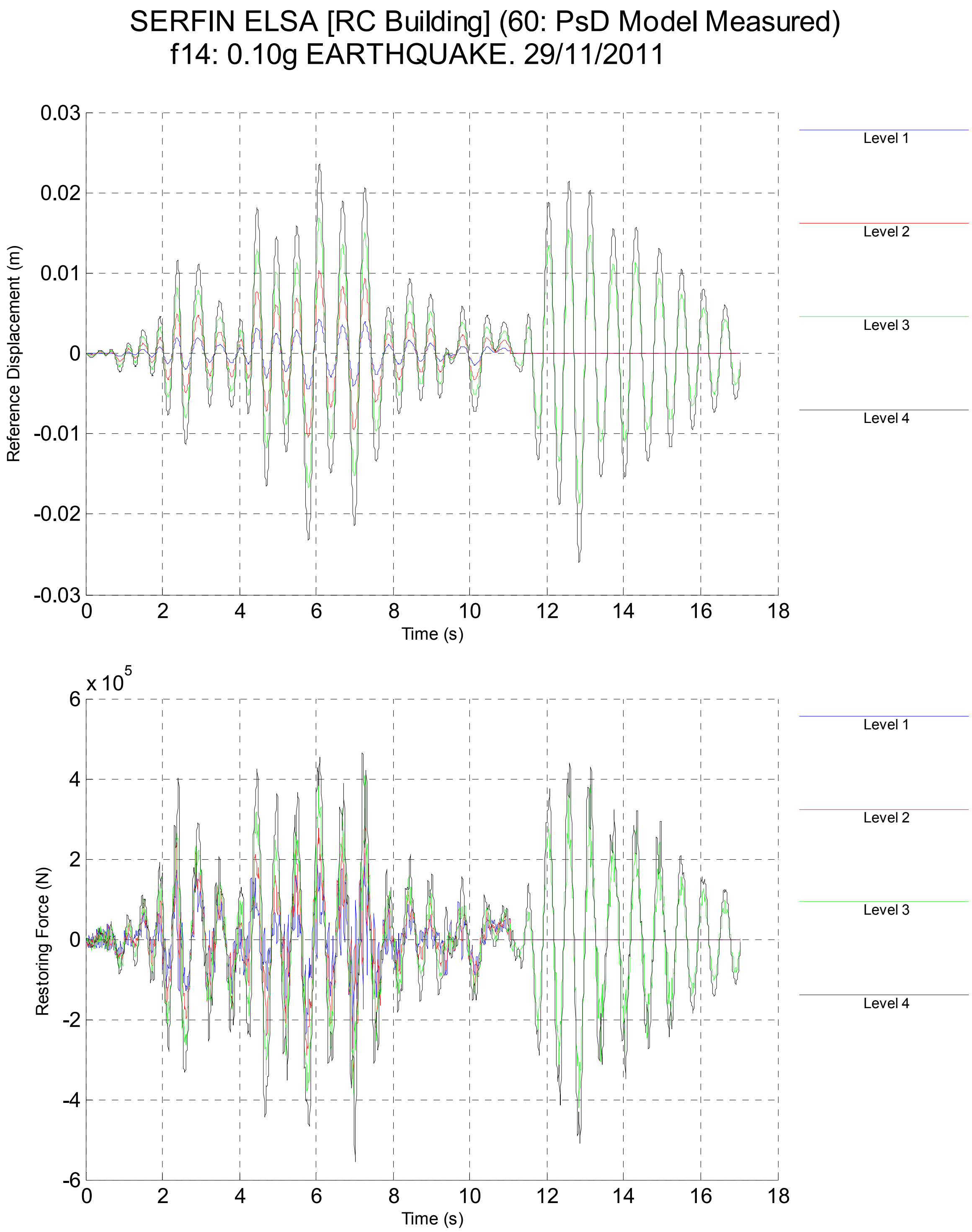

Figure 10.1 0.10g Test - Algorithm Displacement and Force Histories

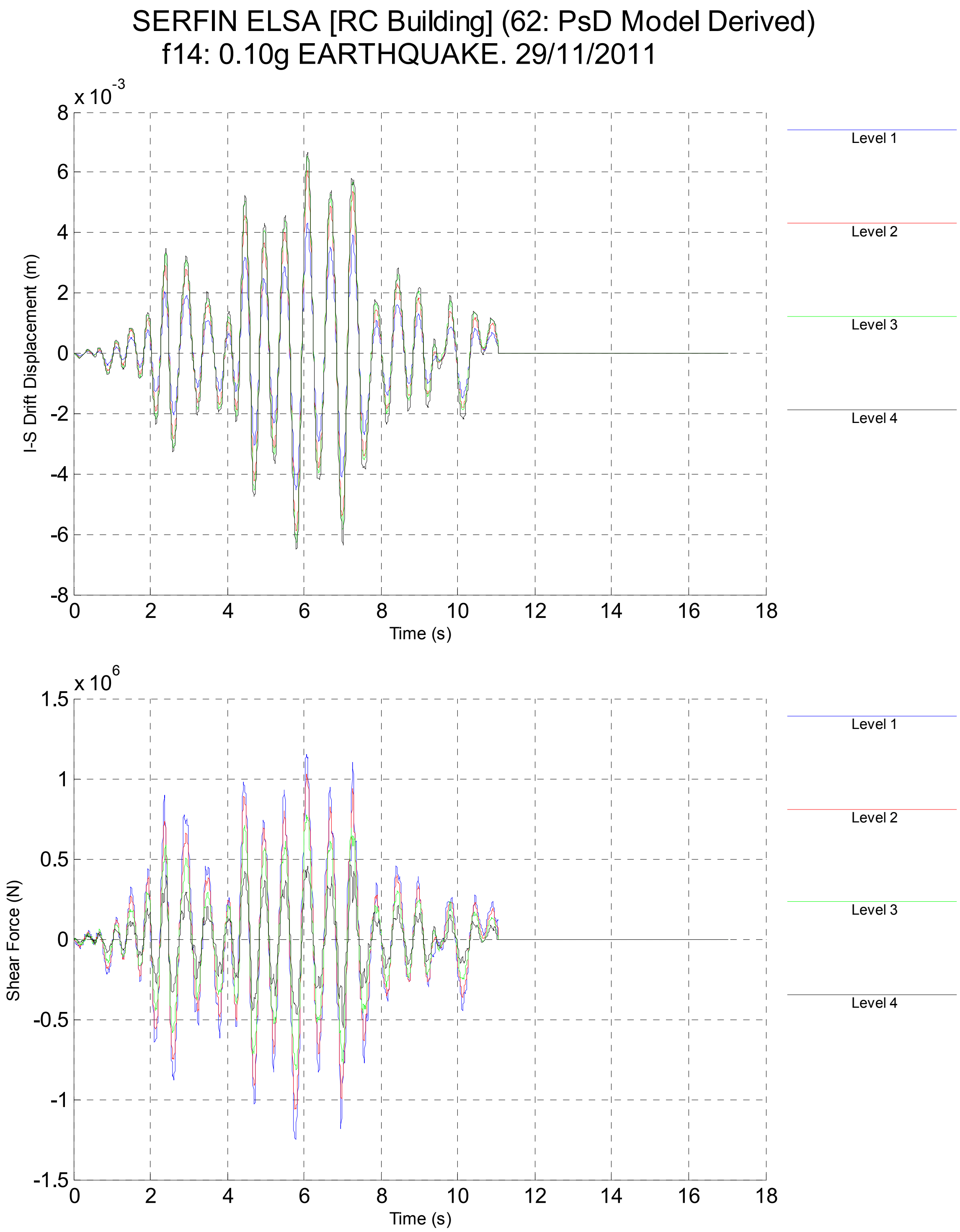

Figure 10.2 0.10g Test - Algorithm Shear Force & Drift Histories

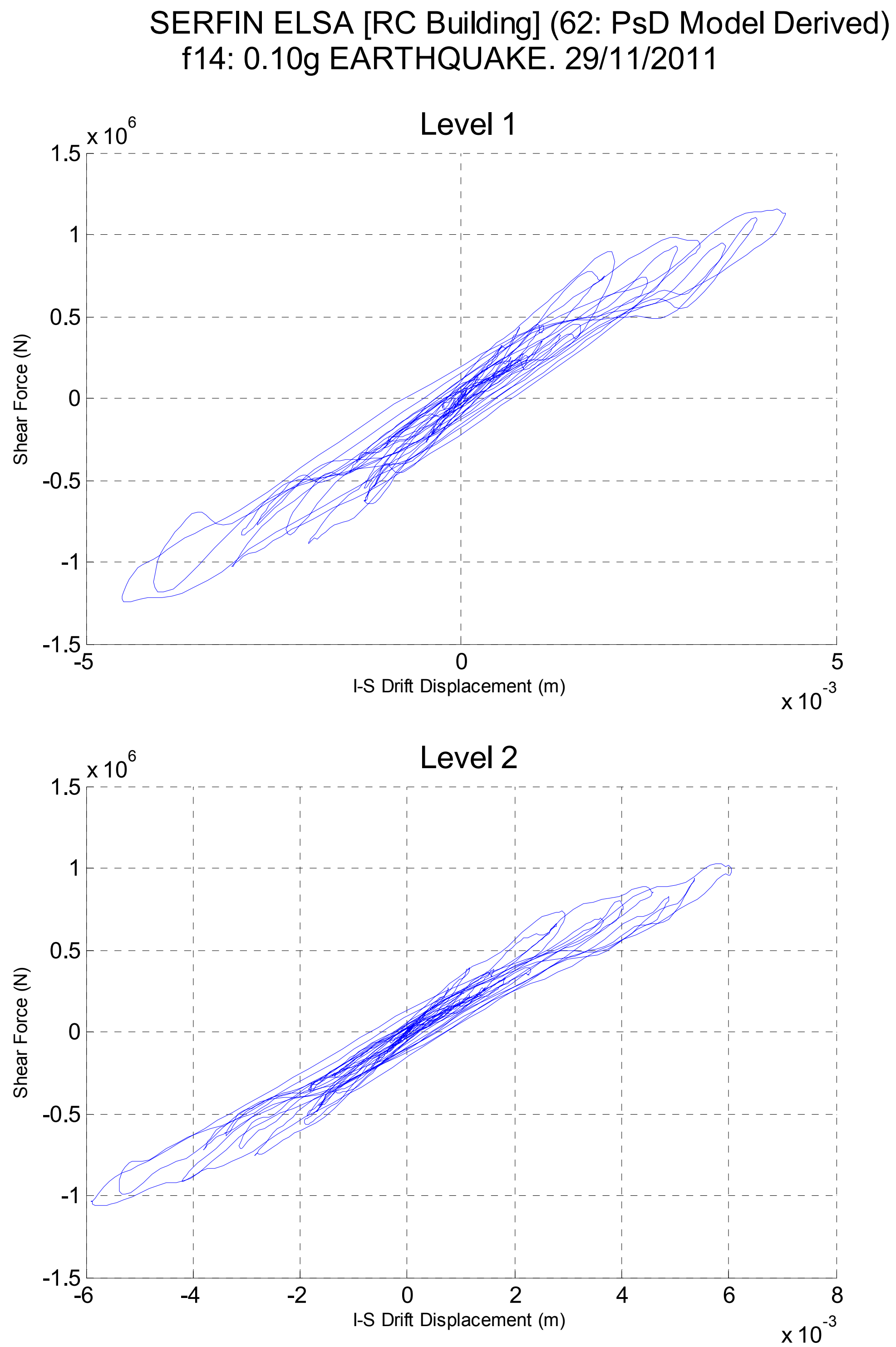

Figure 10.3 0.10g Test - Algorithm Shear Force/Drift Cycle

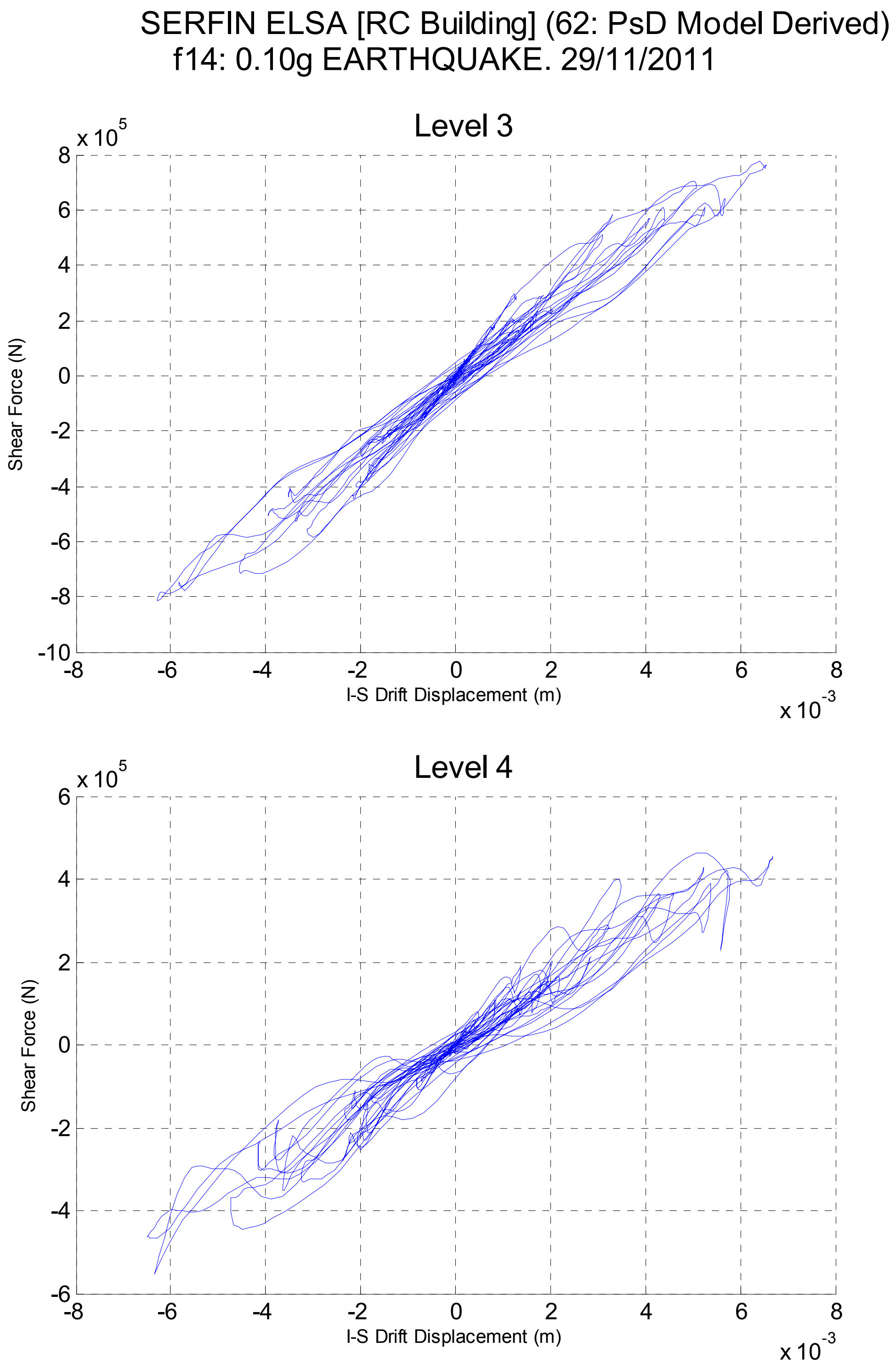

Figure 10.4 0.10g Test - Algorithm Shear Force/Drift Cycle

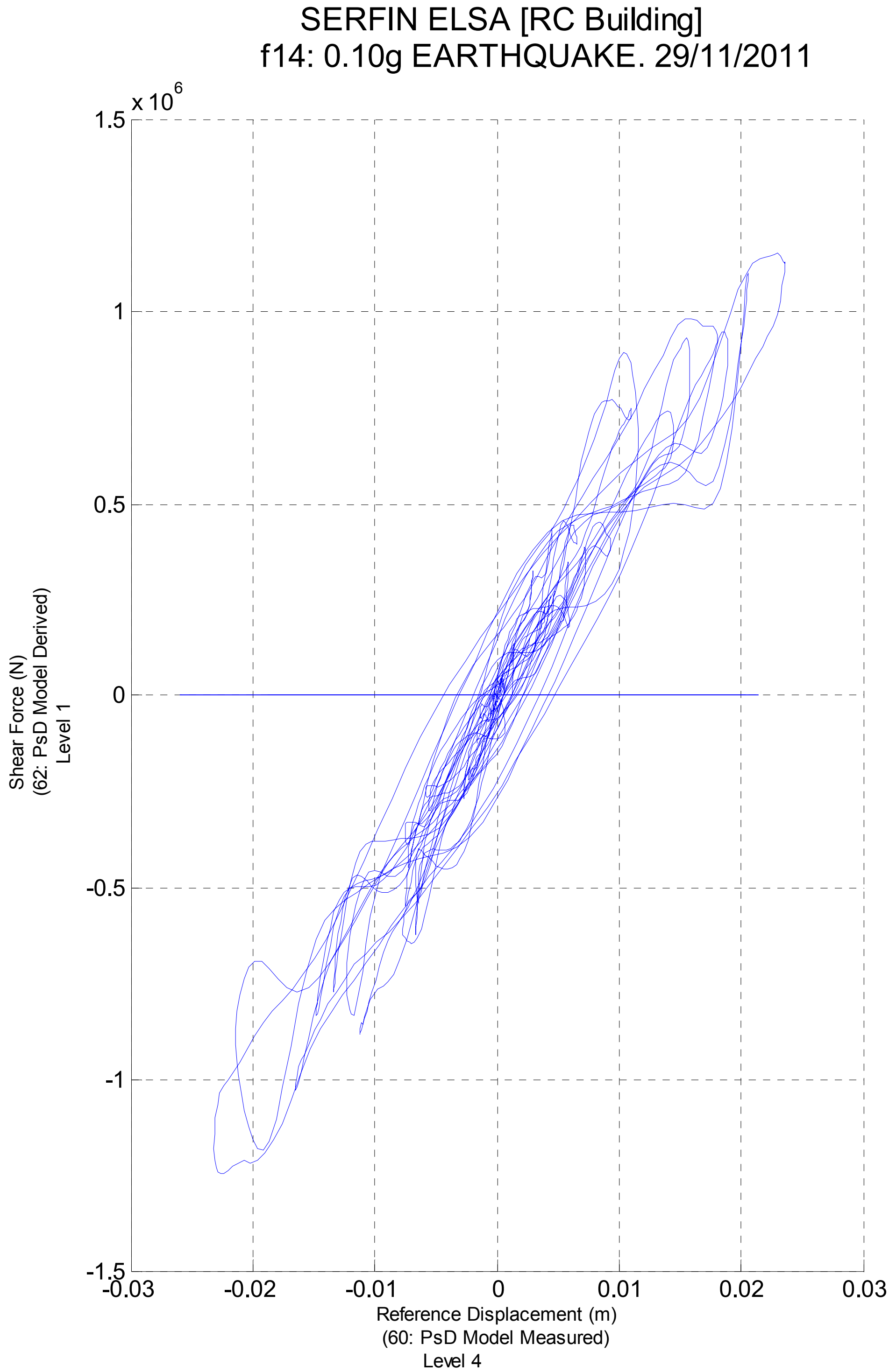

Figure 10.5 0.10g Test - Base Shear Force/Top Displacement Cycle

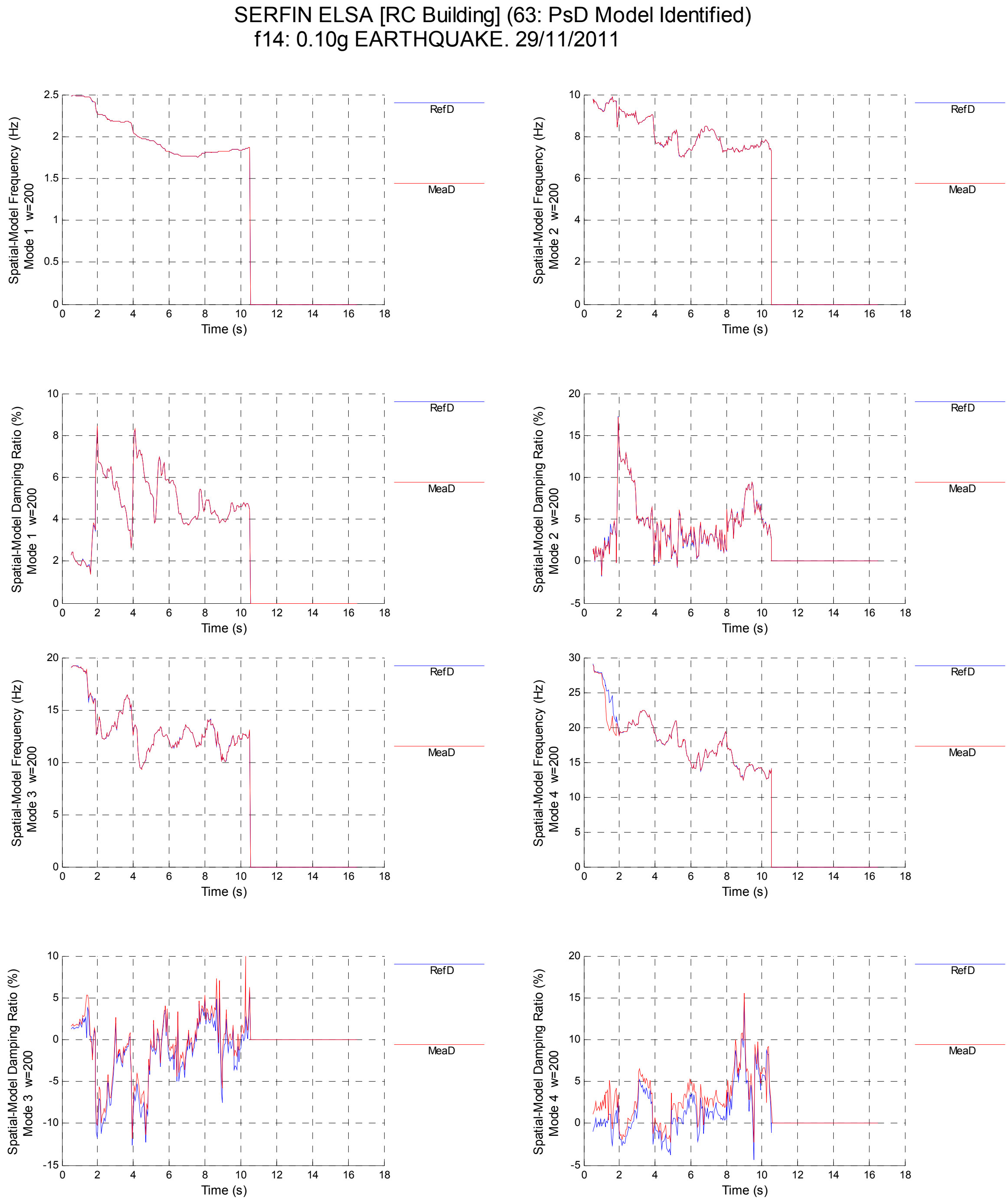

Figure 10.6 0.10g Test - Frequency and damping (measured and performed)

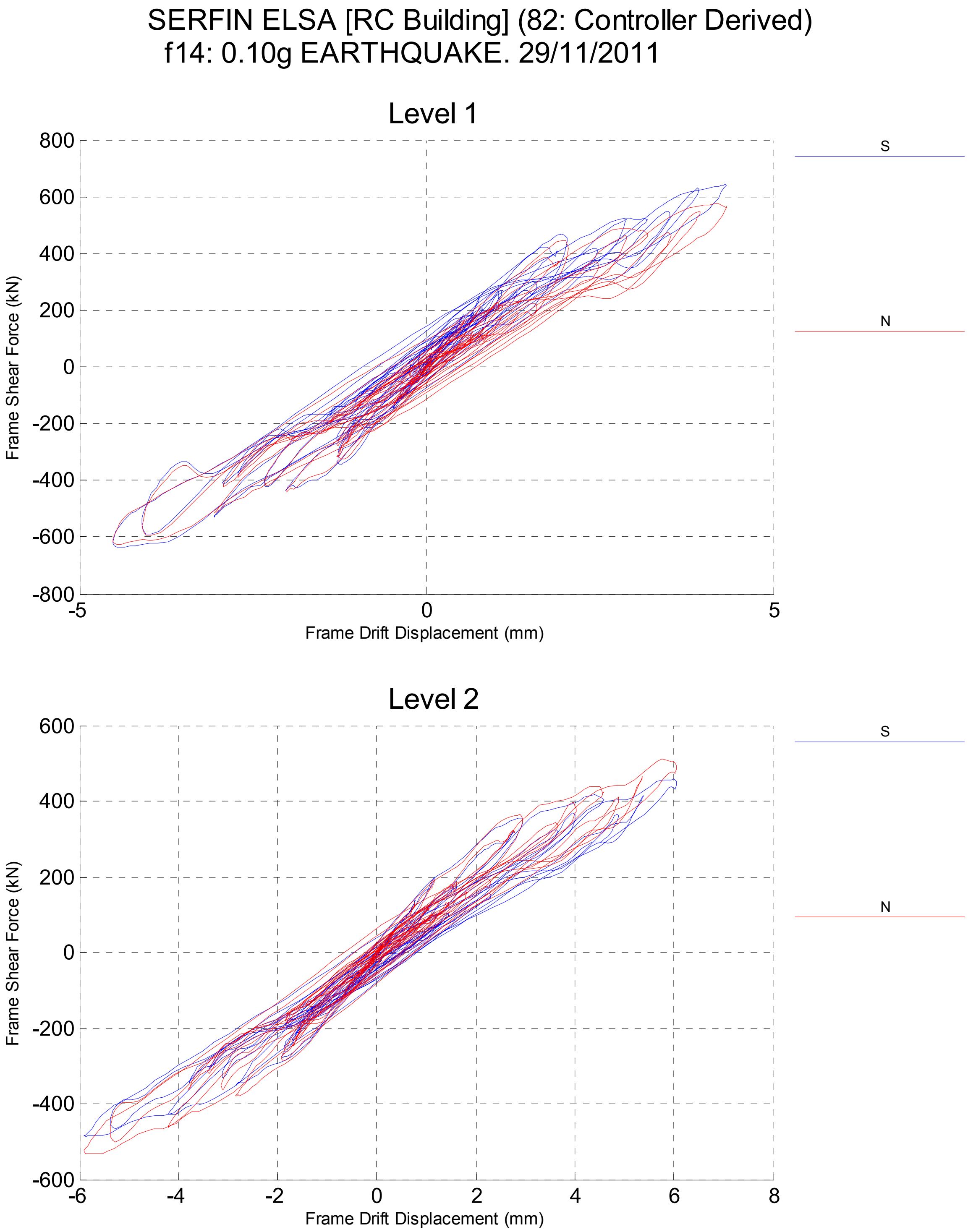

Figure 10.7 0.10g Test - Frame Shear Force/Drift Cycle

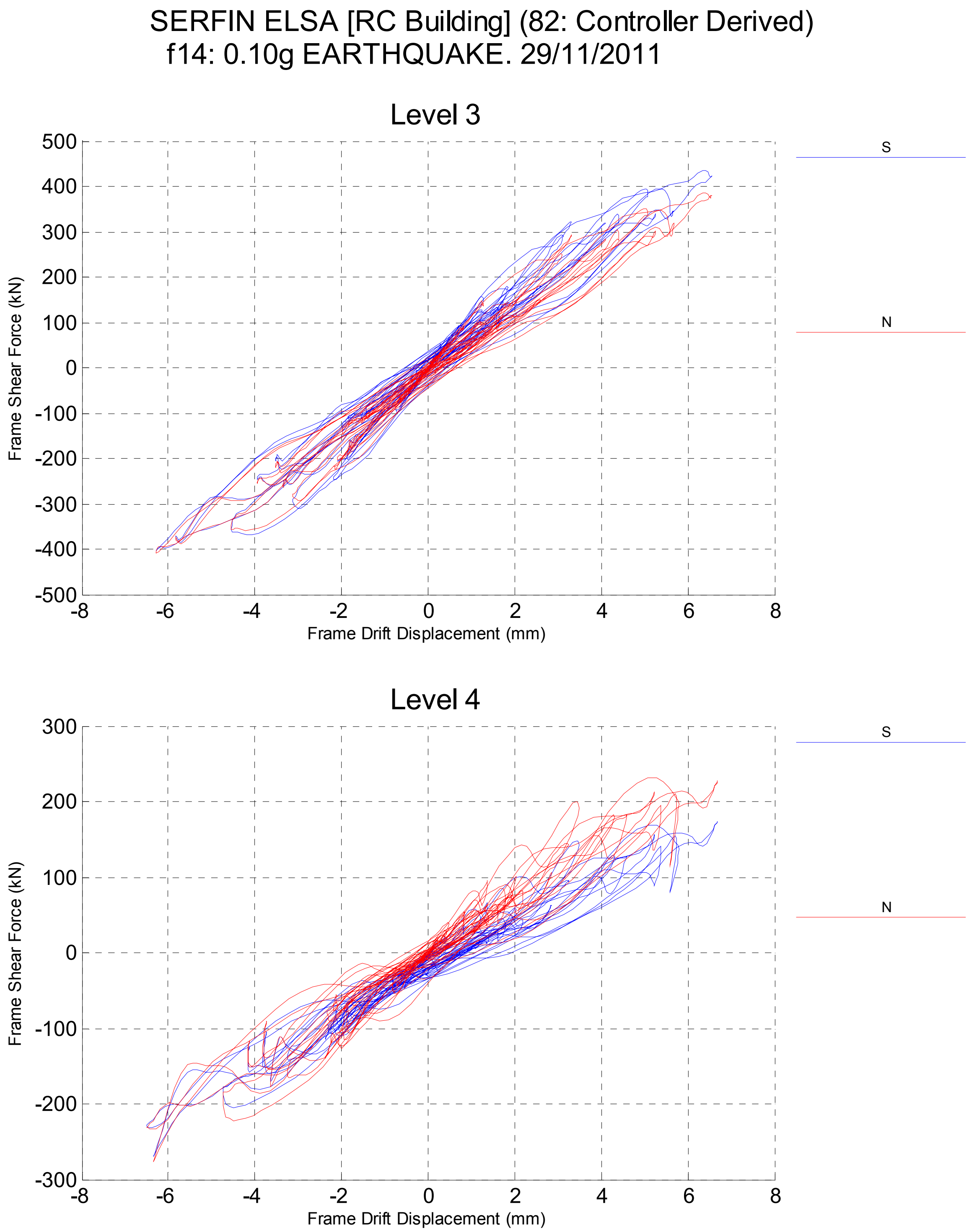

Figure 10.8 0.10g Test - Frame Shear Force/Drift Cycle

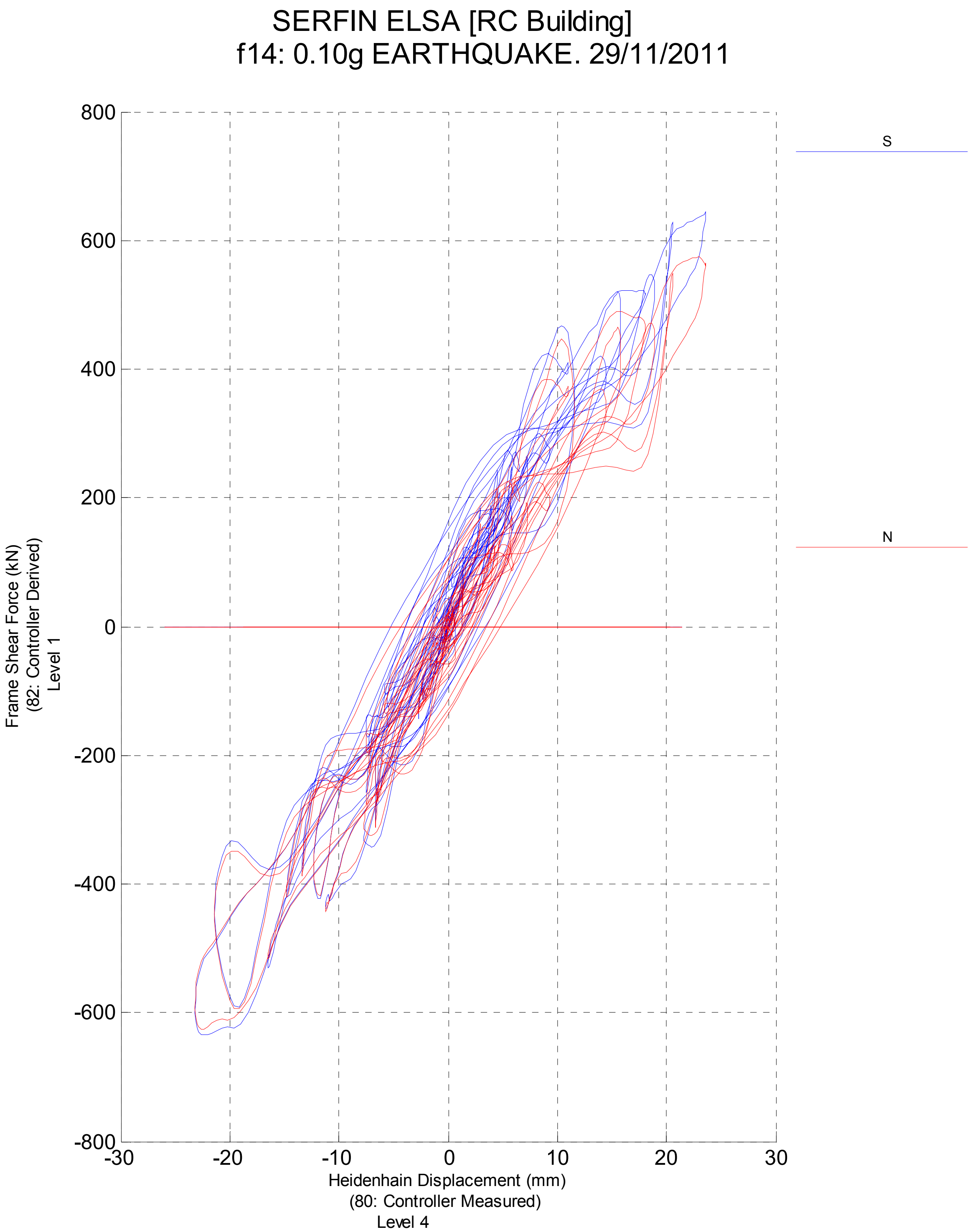

Figure 10.9 0.10g Test - Frame Base Shear/Top Displacement

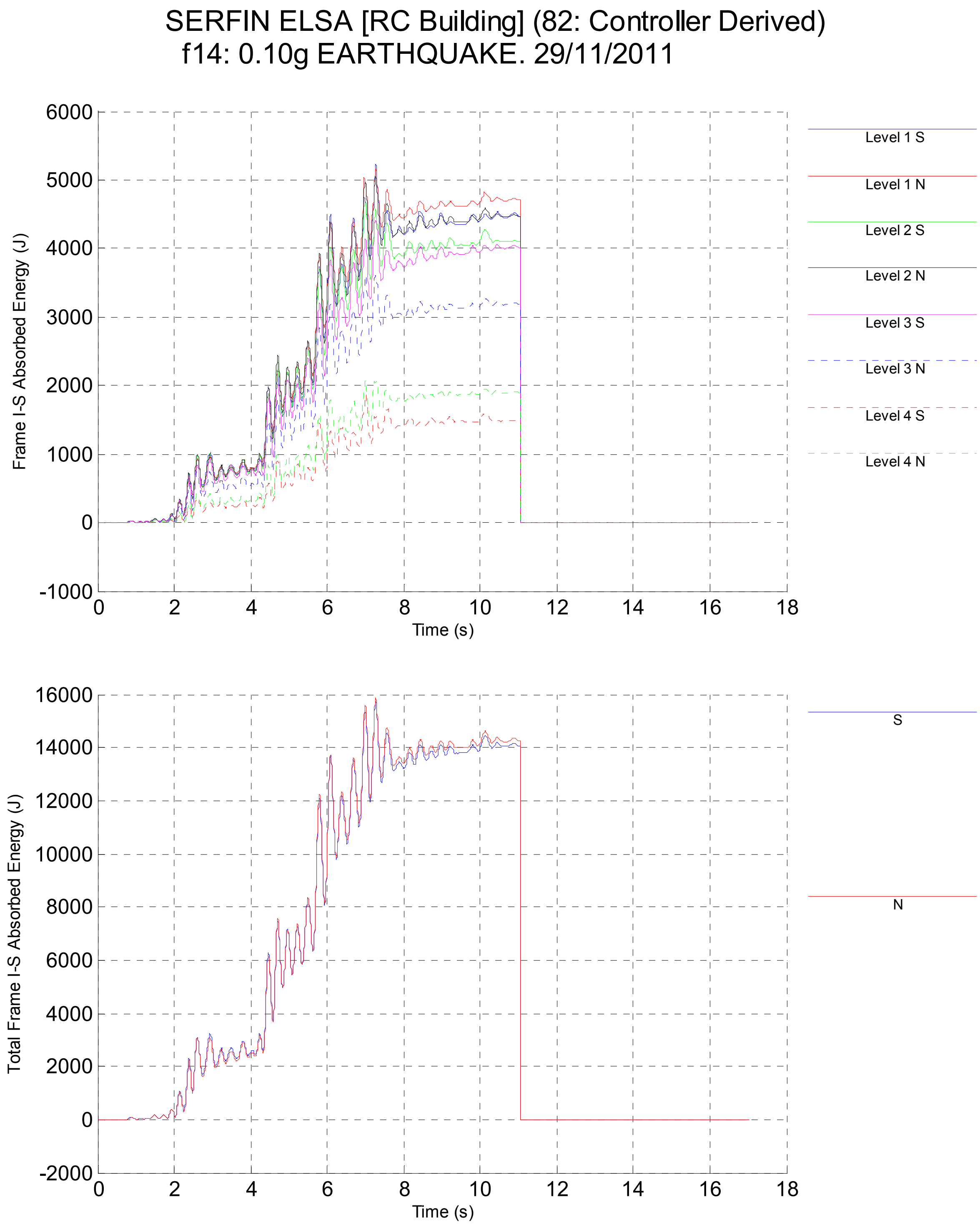

Figure 10.10 0.10g Test - Frame Energy Histories

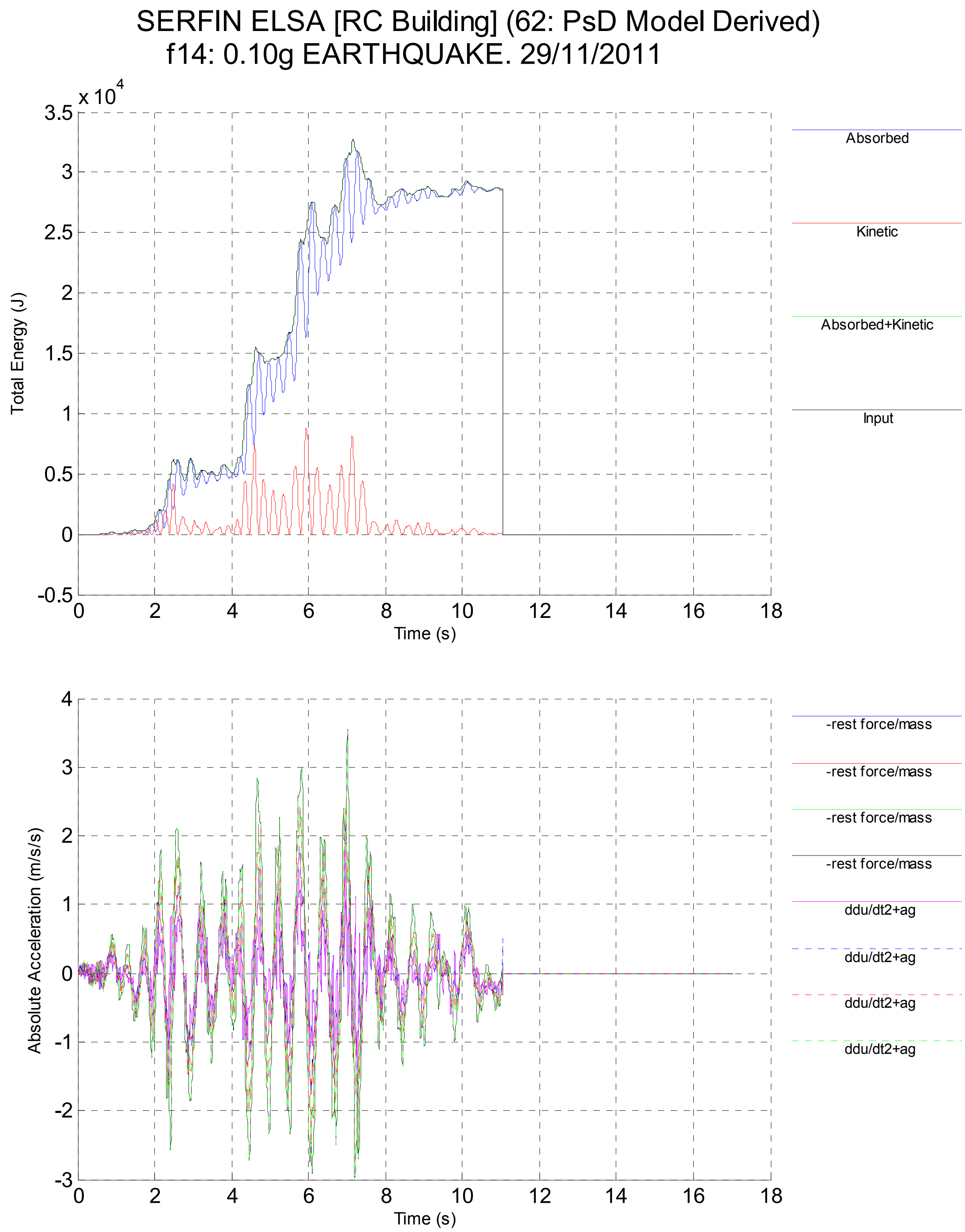

Figure 10.11 0.10g Test - Algorithm Energy and Acc. Histories

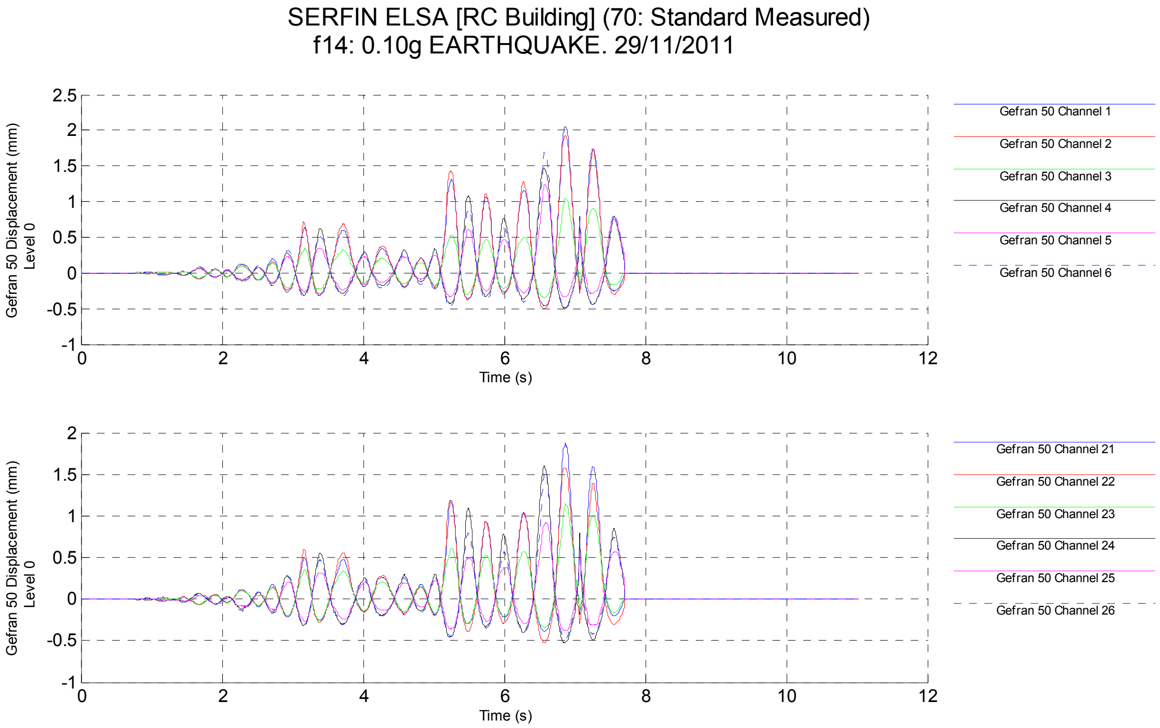

Figure 10.12 0.10g Test - Local Displacements - Ground Floor

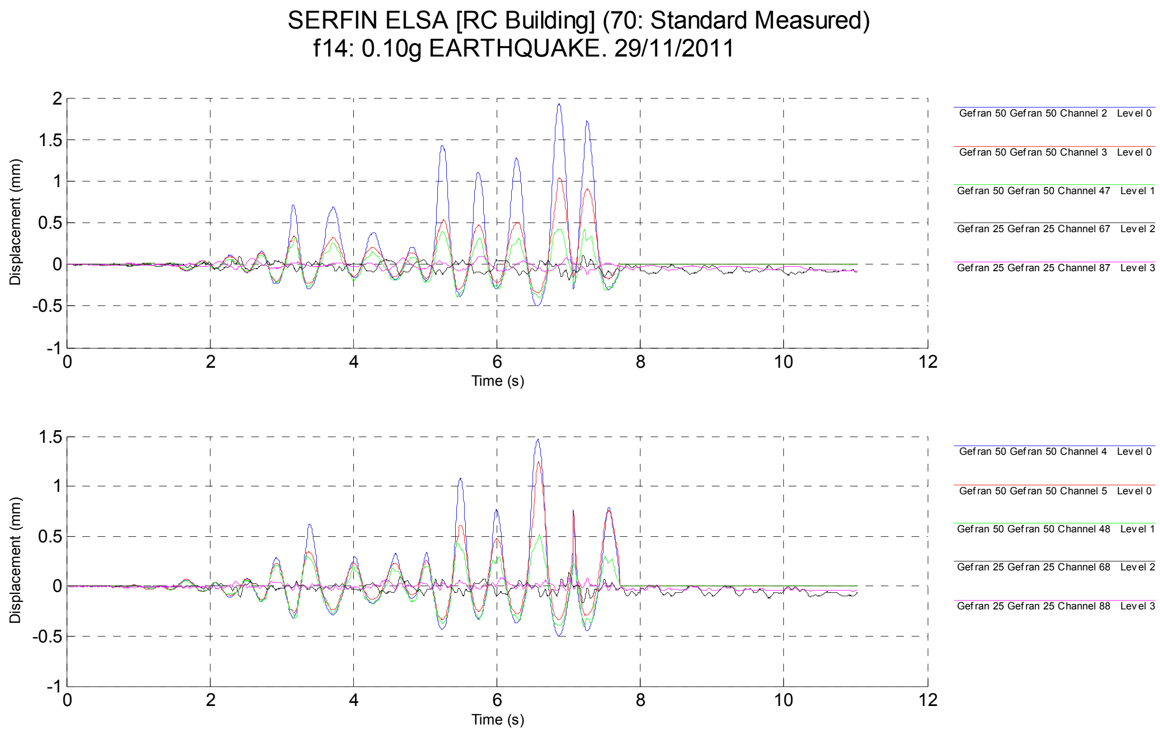

Figure 10.13 0.10g Test - Local Displacements - South Wall Columns

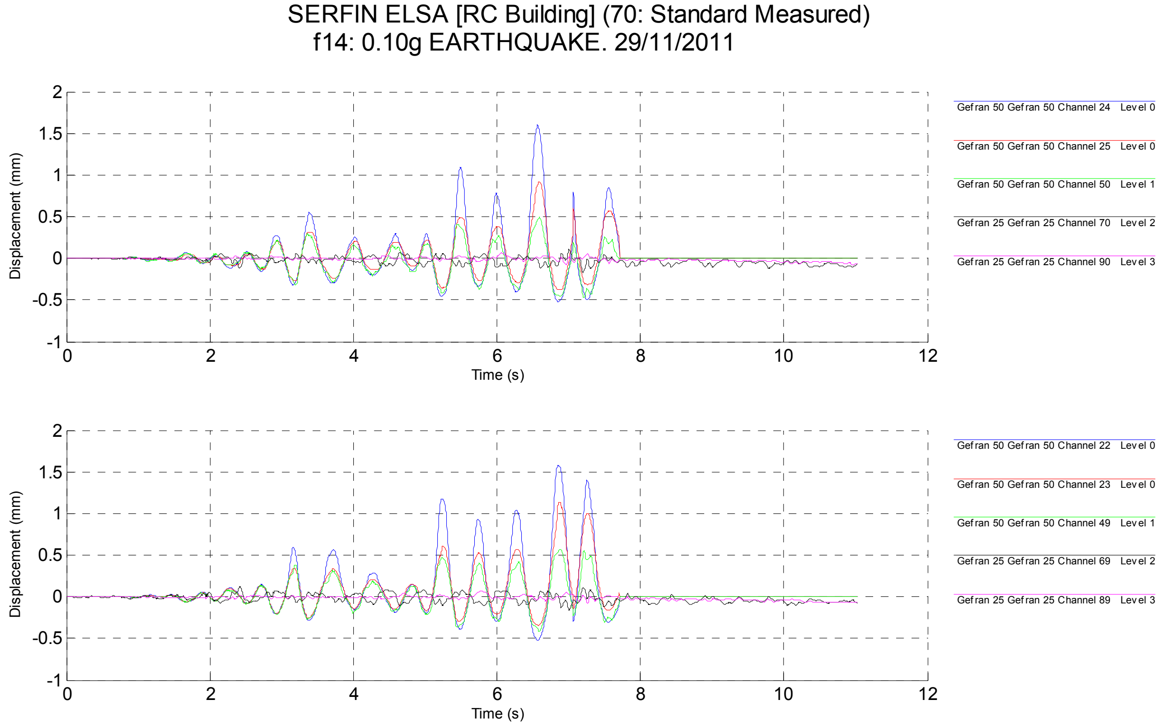

Figure 10.14 0.10g Test - Local Displacements - North Wall Columns

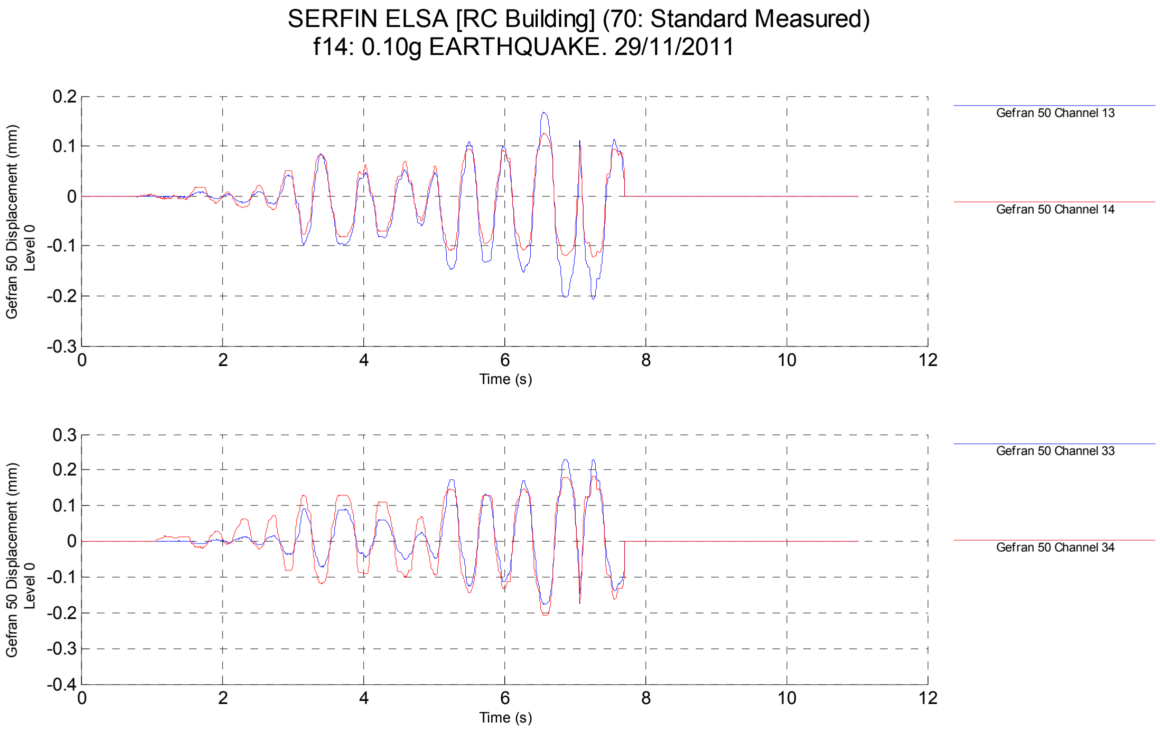

Figure 10.15 0.10g Test - Local Slip Displacements - Ground Beam

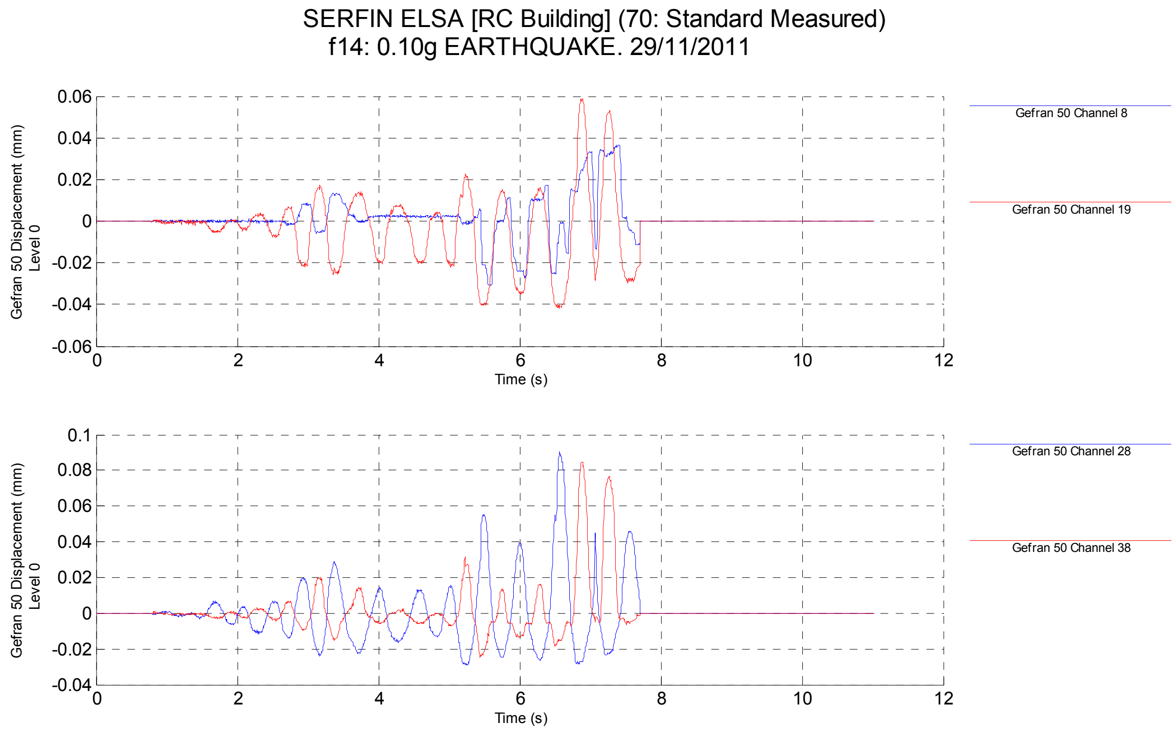

Figure 10.16 0.10g Test - Local Slip Displacements - Ground Column

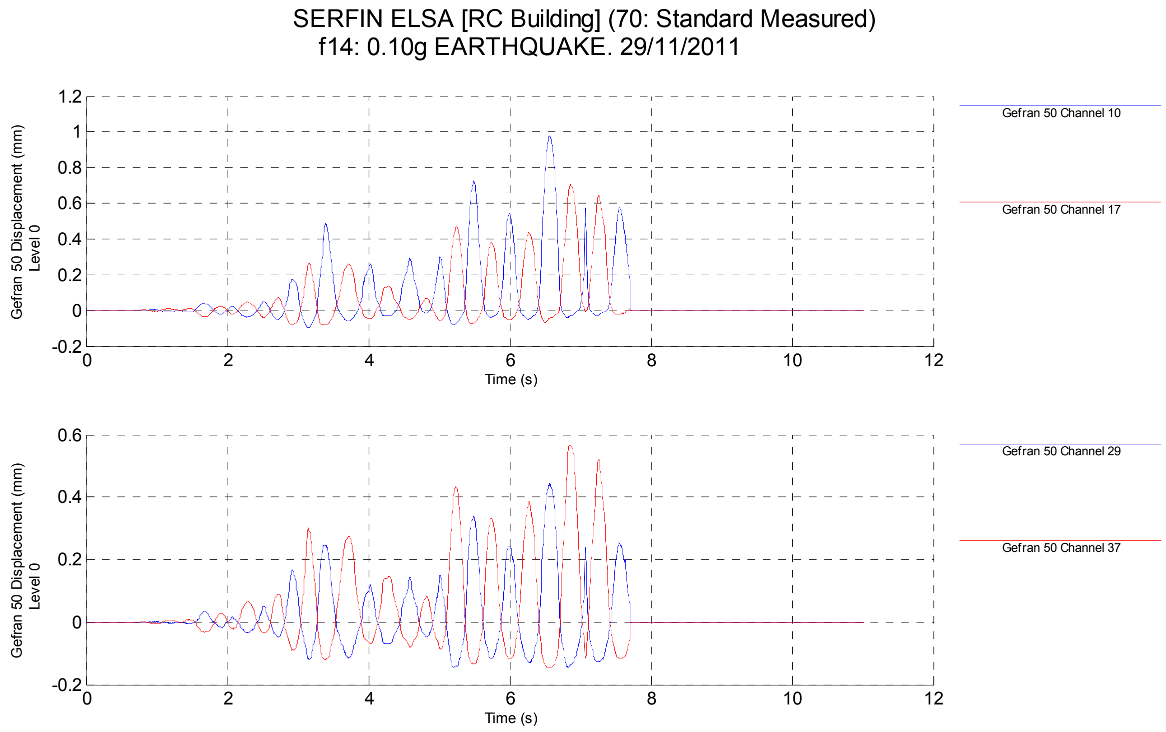

Figure 10.17 0.10g Test - Local Bottom Opening of the Wall, Ground Floor

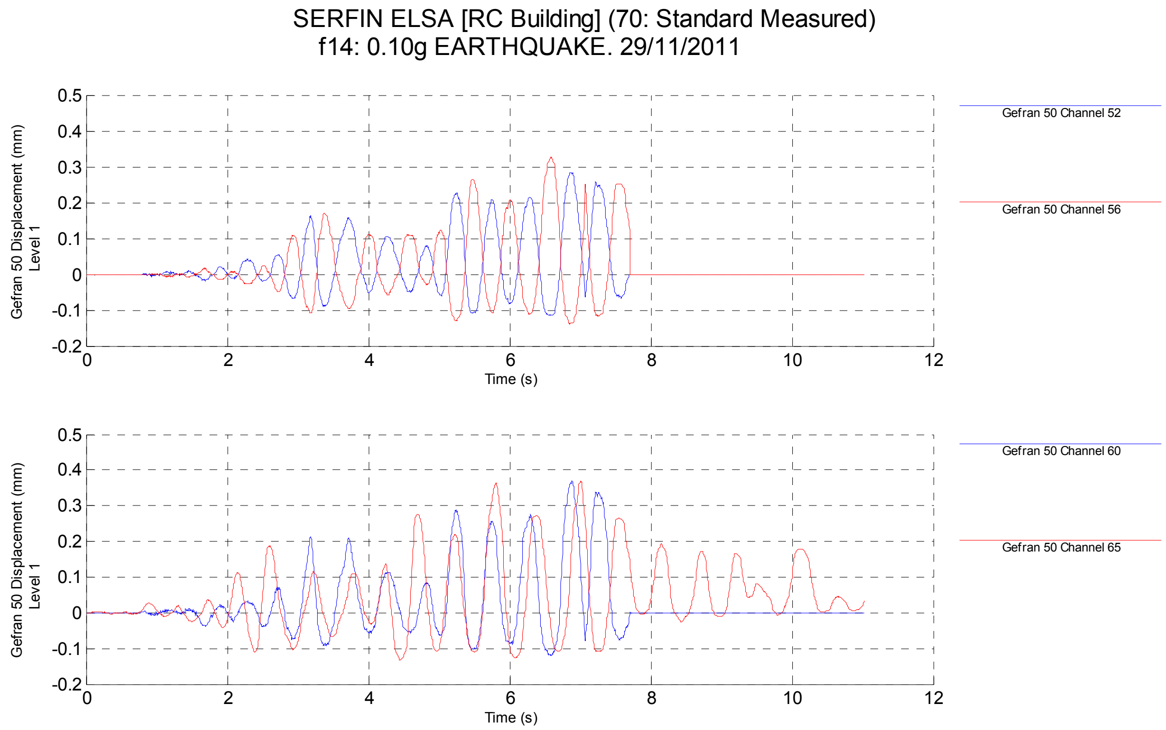

Figure 10.18 0.10g Test - Local Bottom Opening of the Wall, 1st Floor

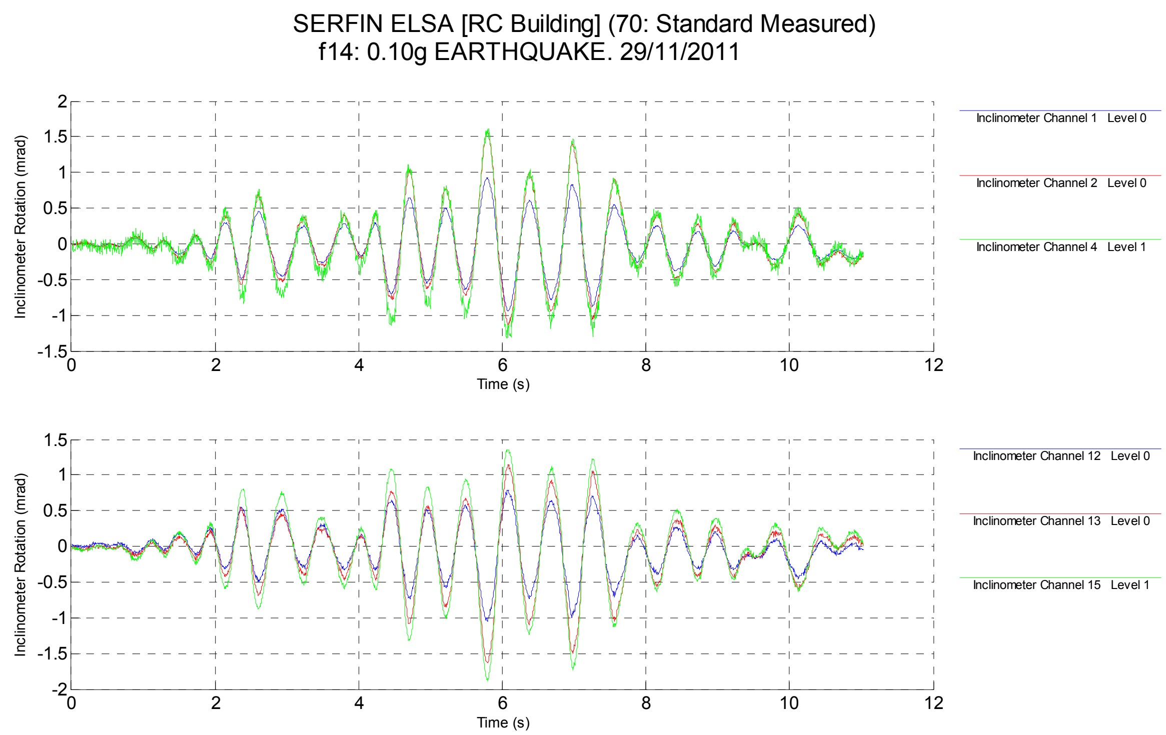

Figure 10.19 0.10g Test - Rotations - West Column

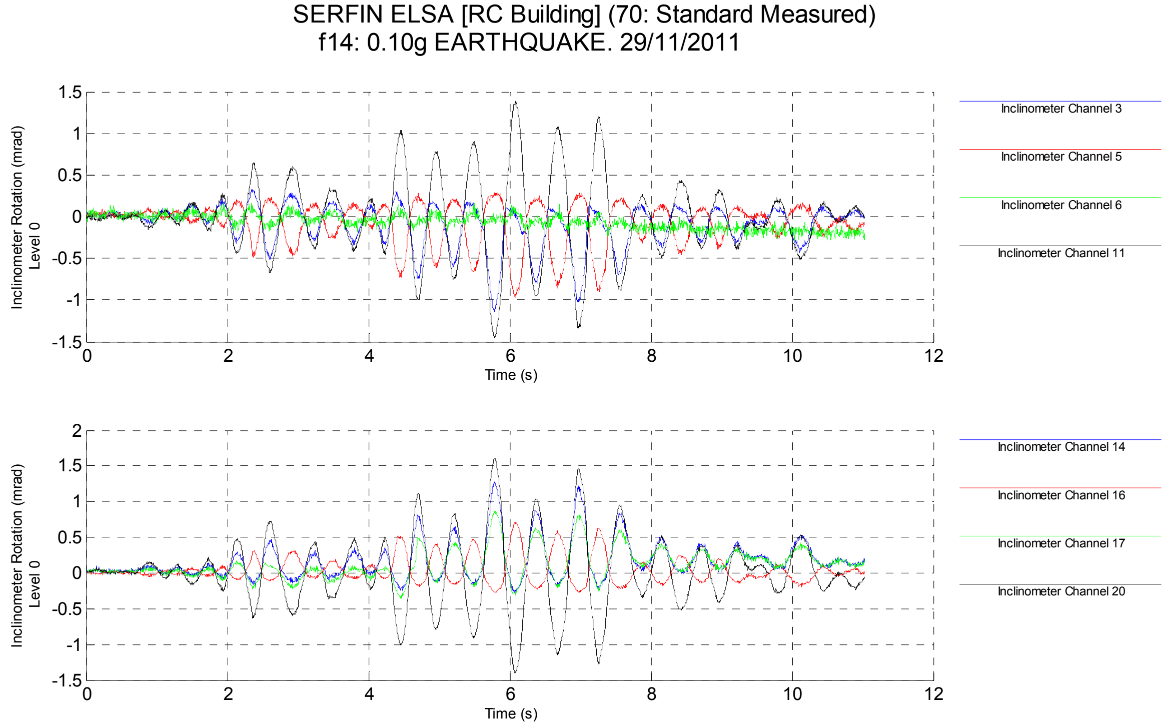

Figure 10.20 0.10g Test - Rotations - 1st Slab

0.25G TEST

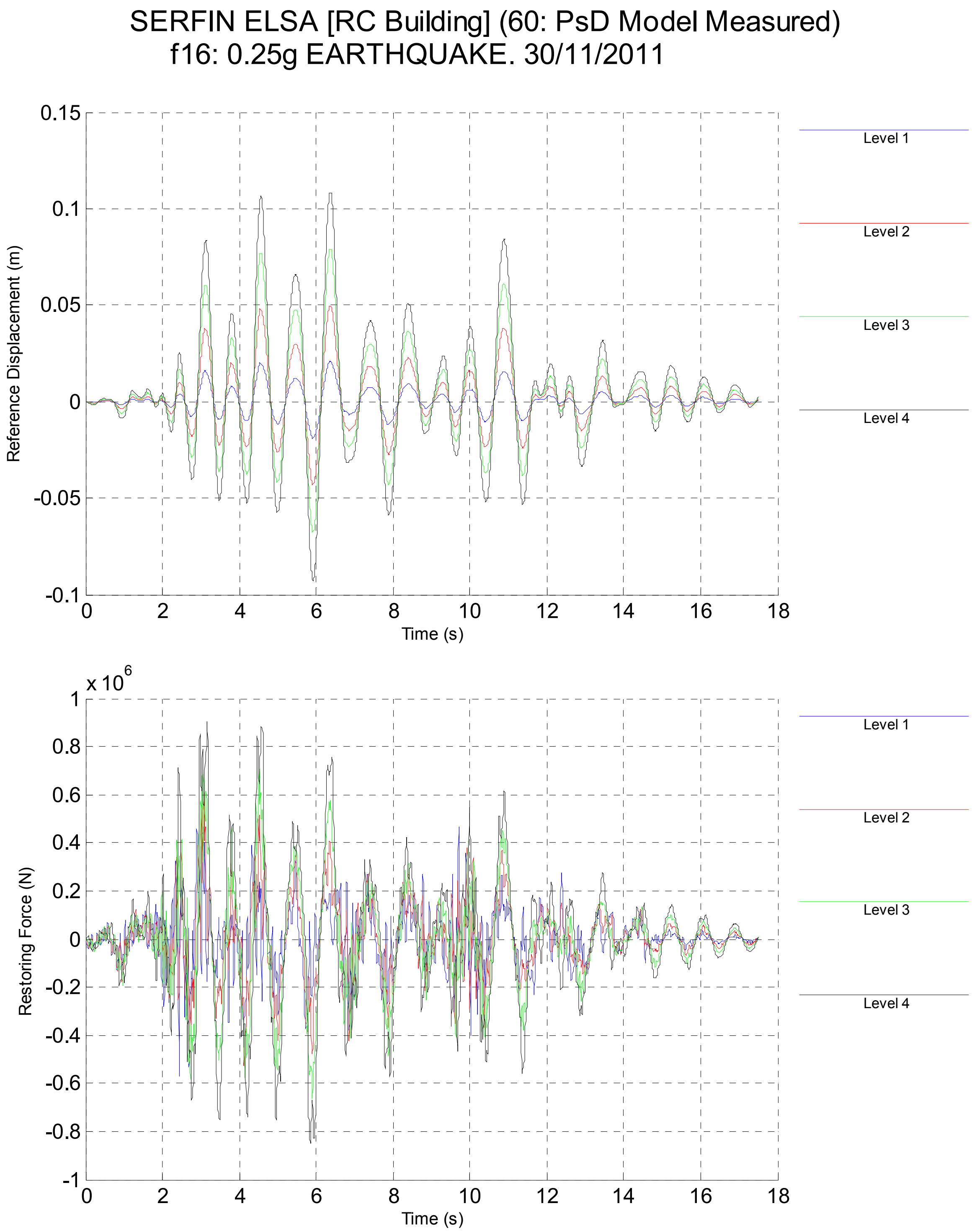

Figure 10.21 0.25g Test - Algorithm Displacement and Force Histories

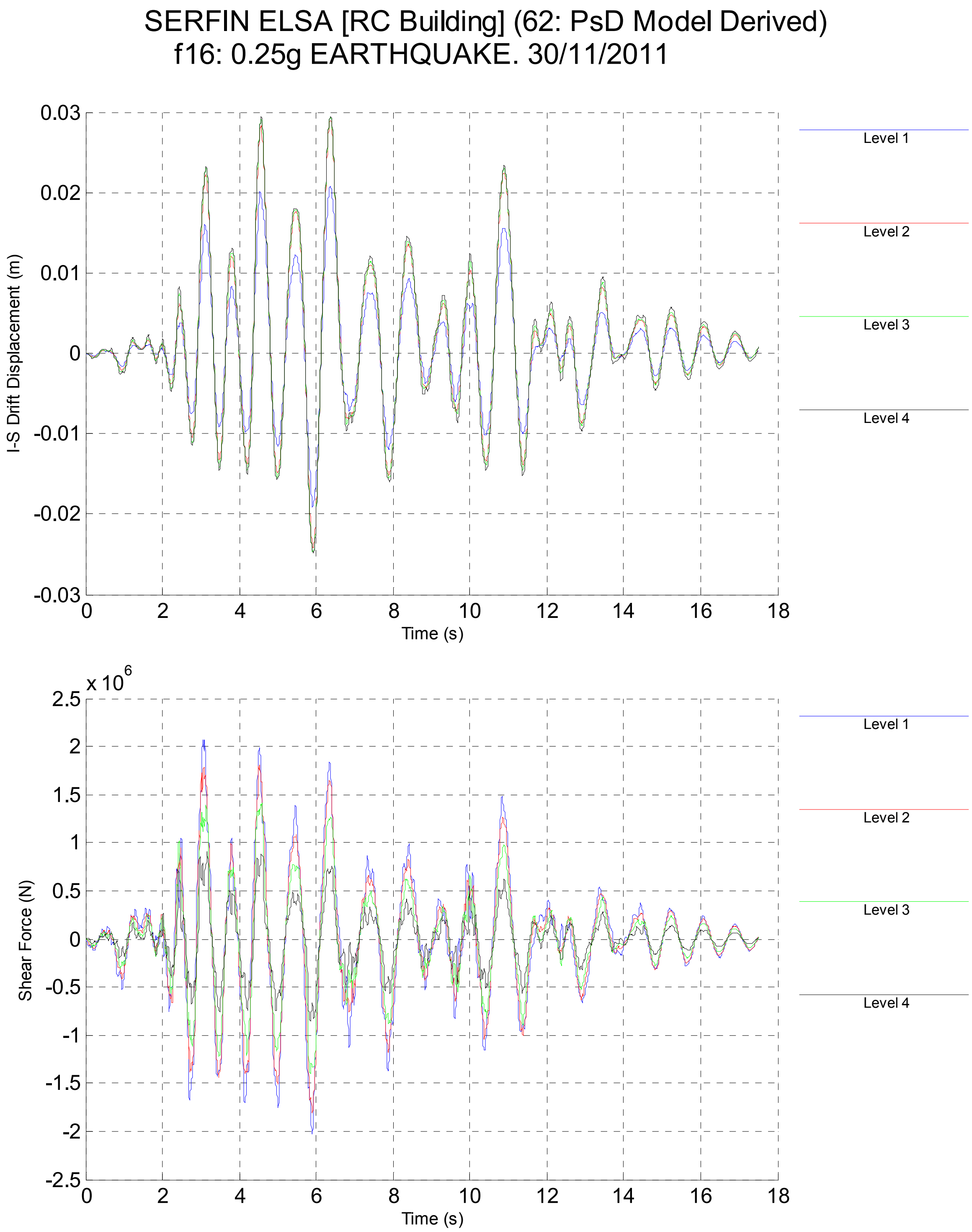

Figure 10.22 0.25g Test - Algorithm Shear Force & Drift Histories

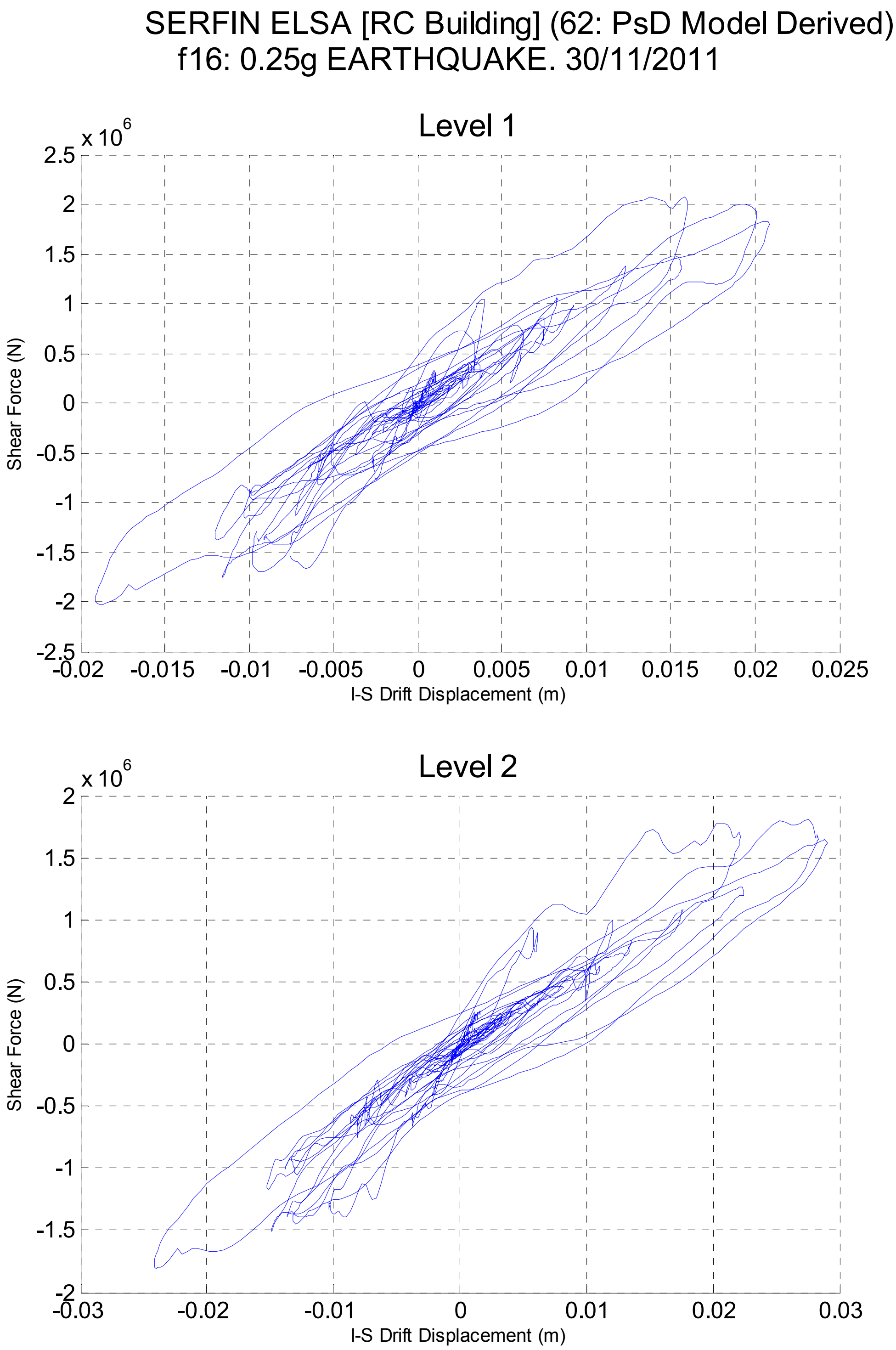

Figure 10.23 0.25g Test - Algorithm Shear Force/Drift Cycle

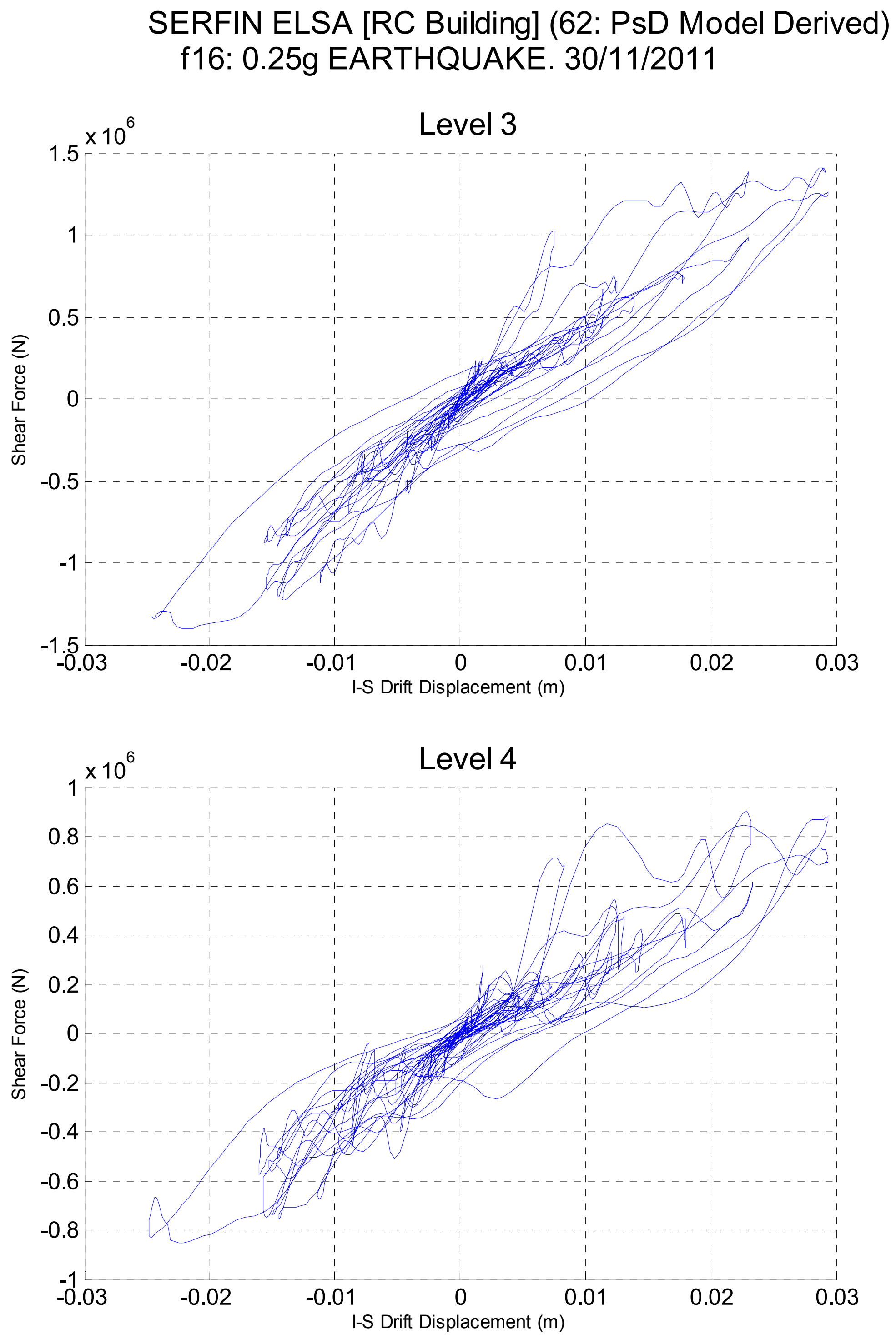

Figure 10.24 0.25g Test - Algorithm Shear Force/Drift Cycle

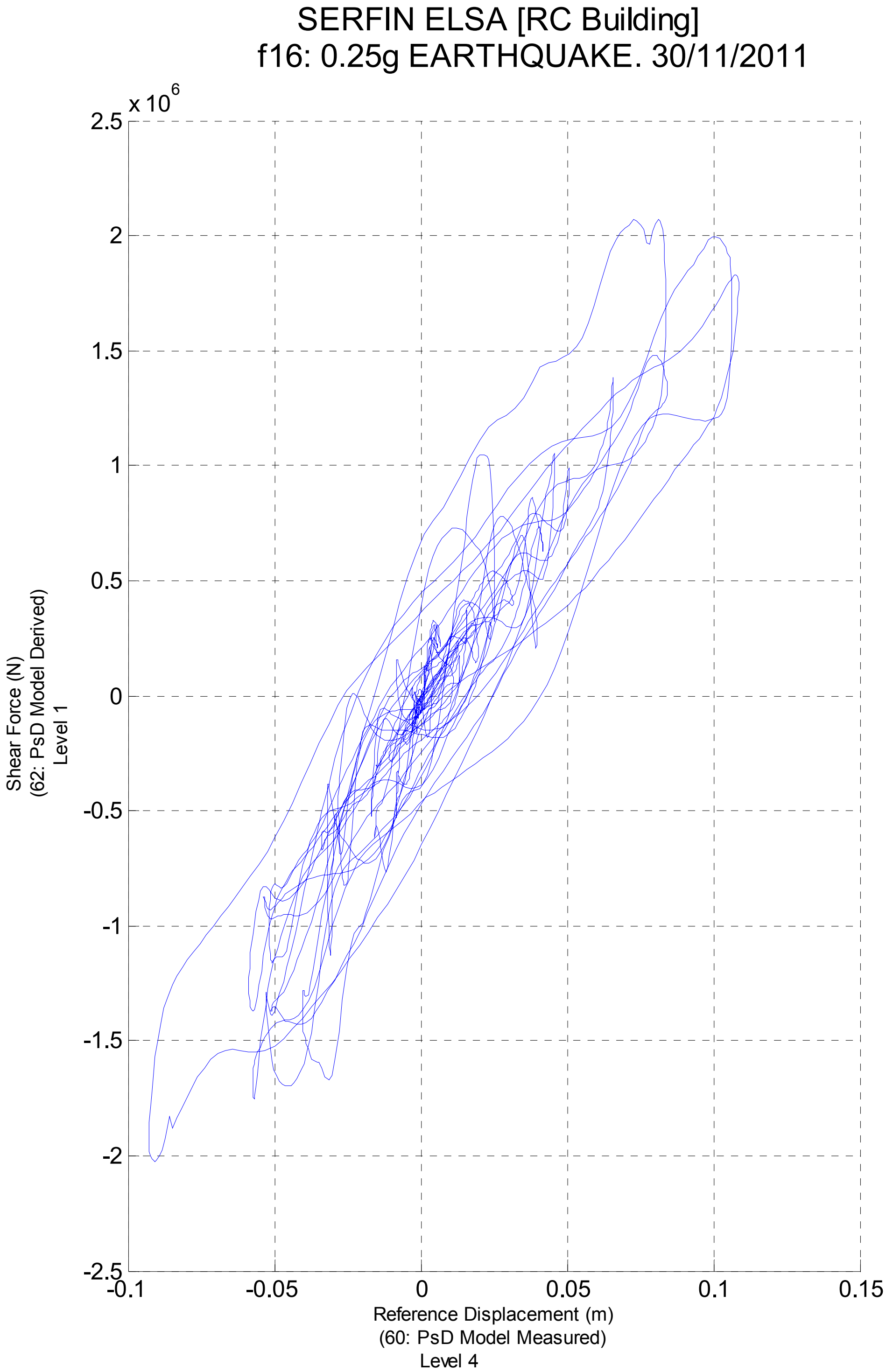

Figure 10.25 0.25g Test - Base Shear Force/Top Displacement Cycle

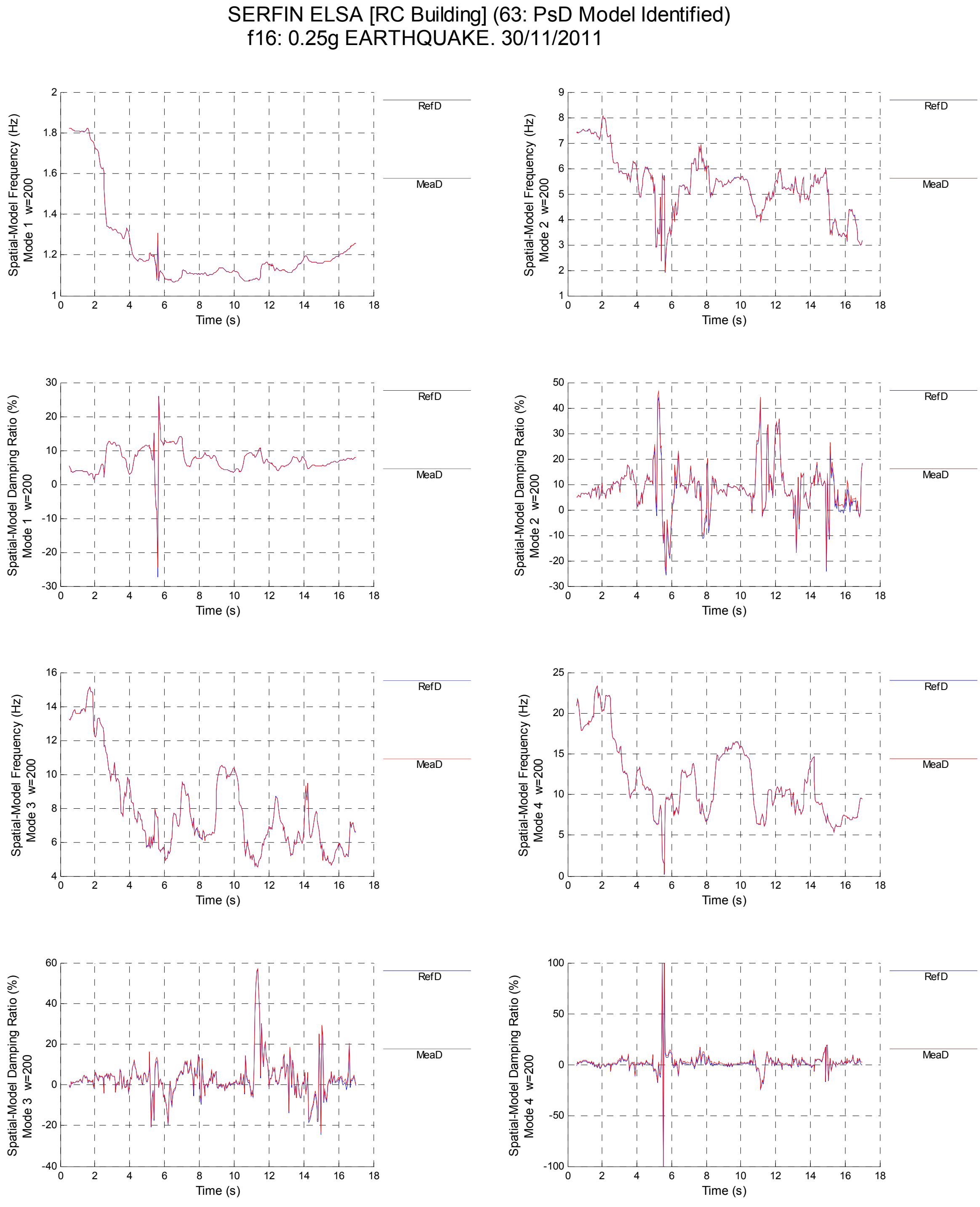

Figure 10.26 0.25g Test - Frequency and damping (measured and performed)

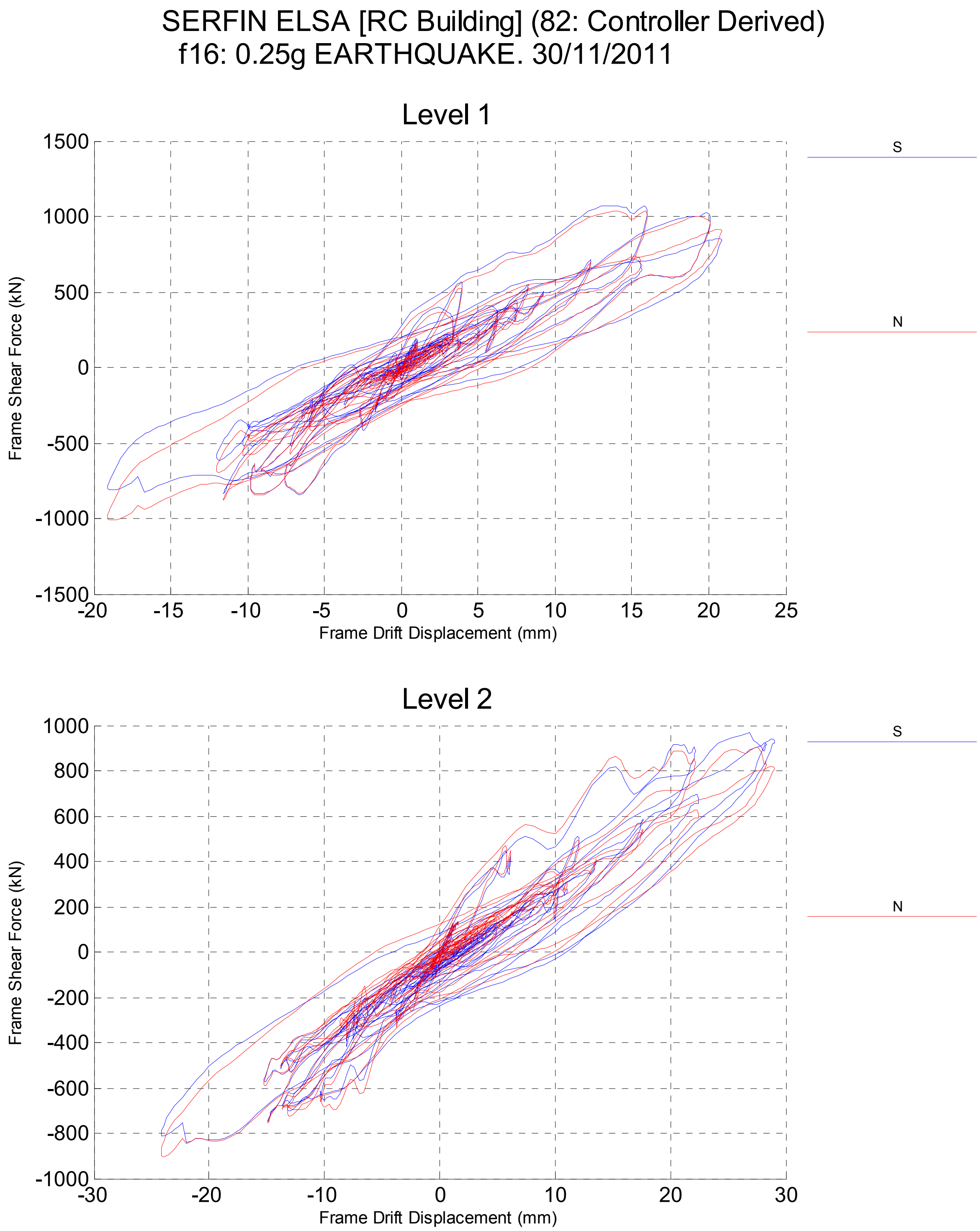

Figure 10.27 0.25g Test - Frame Shear Force/Drift Cycle

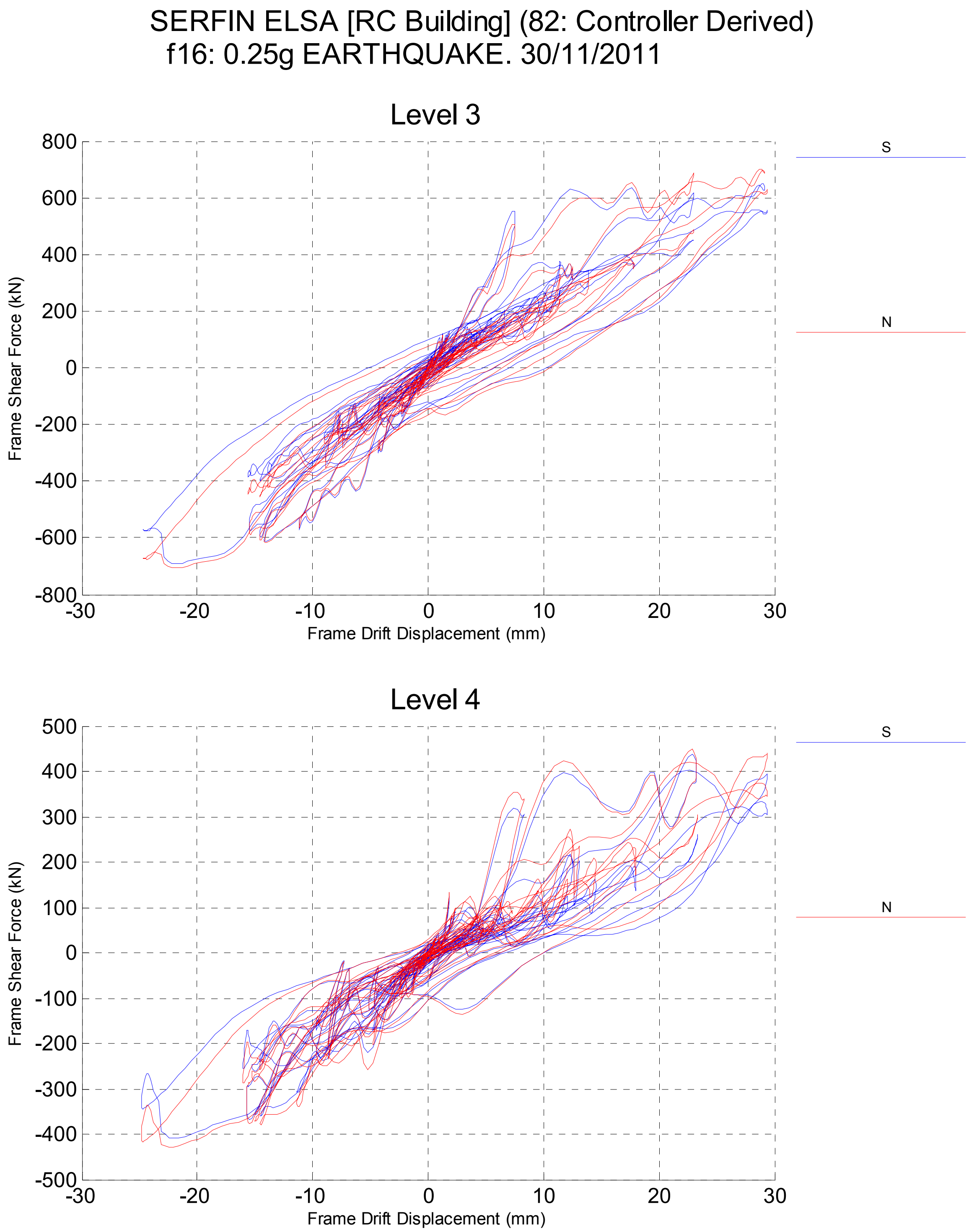

Figure 10.28 0.25g Test - Frame Shear Force/Drift Cycle

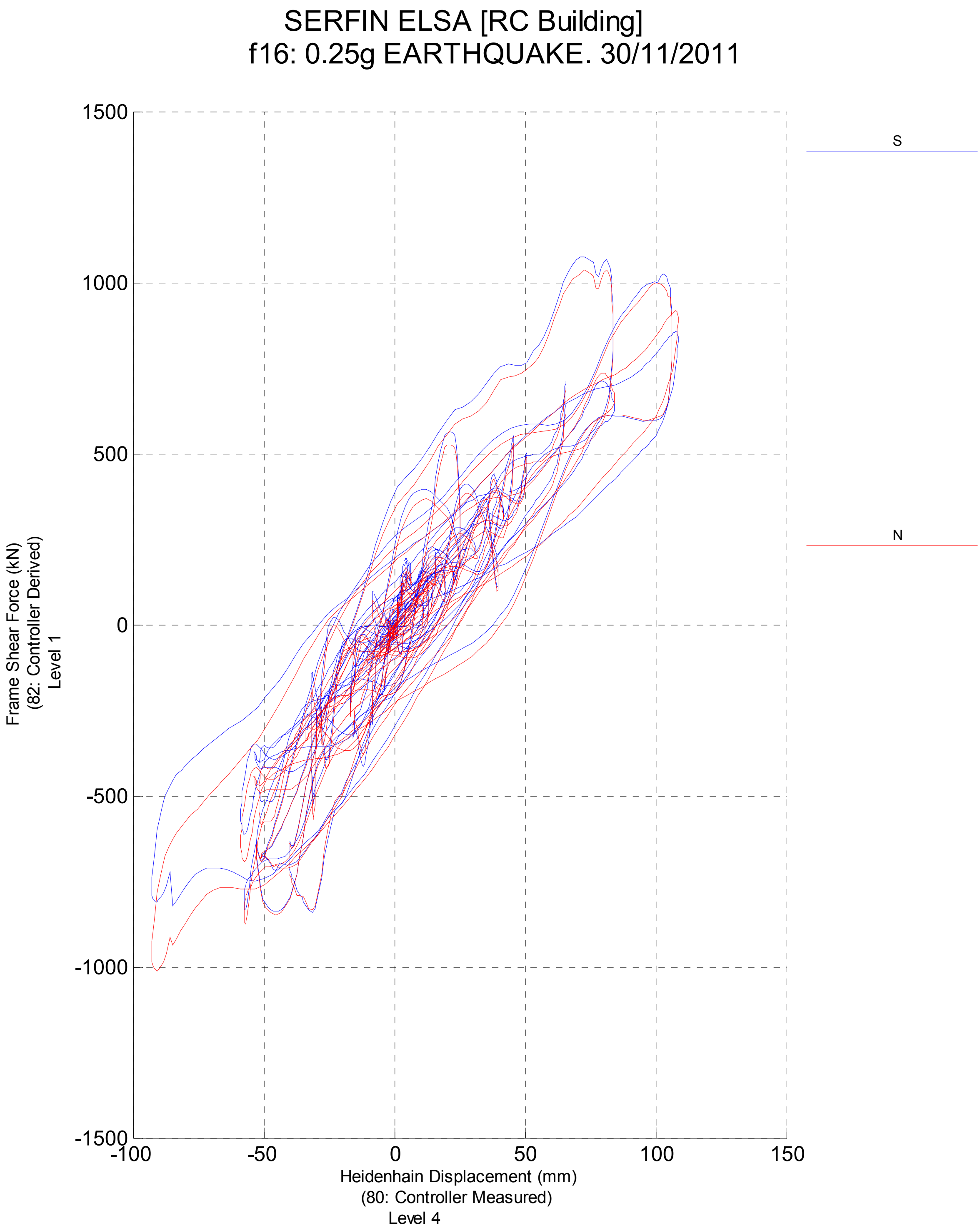

Figure 10.29 0.25g Test - Frame Base Shear/Top Displacement

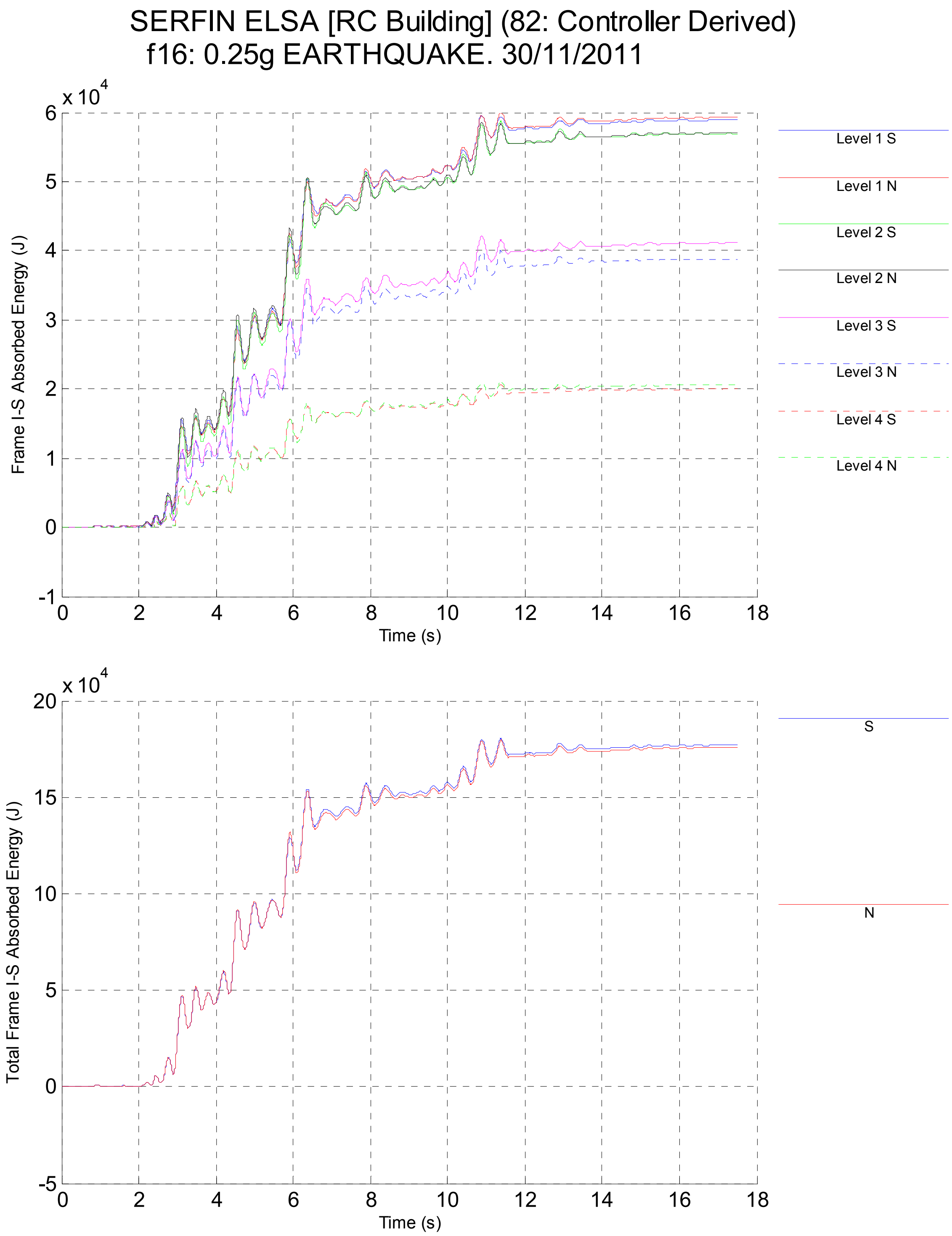

Figure 10.30 0.25g Test - Frame Energy Histories

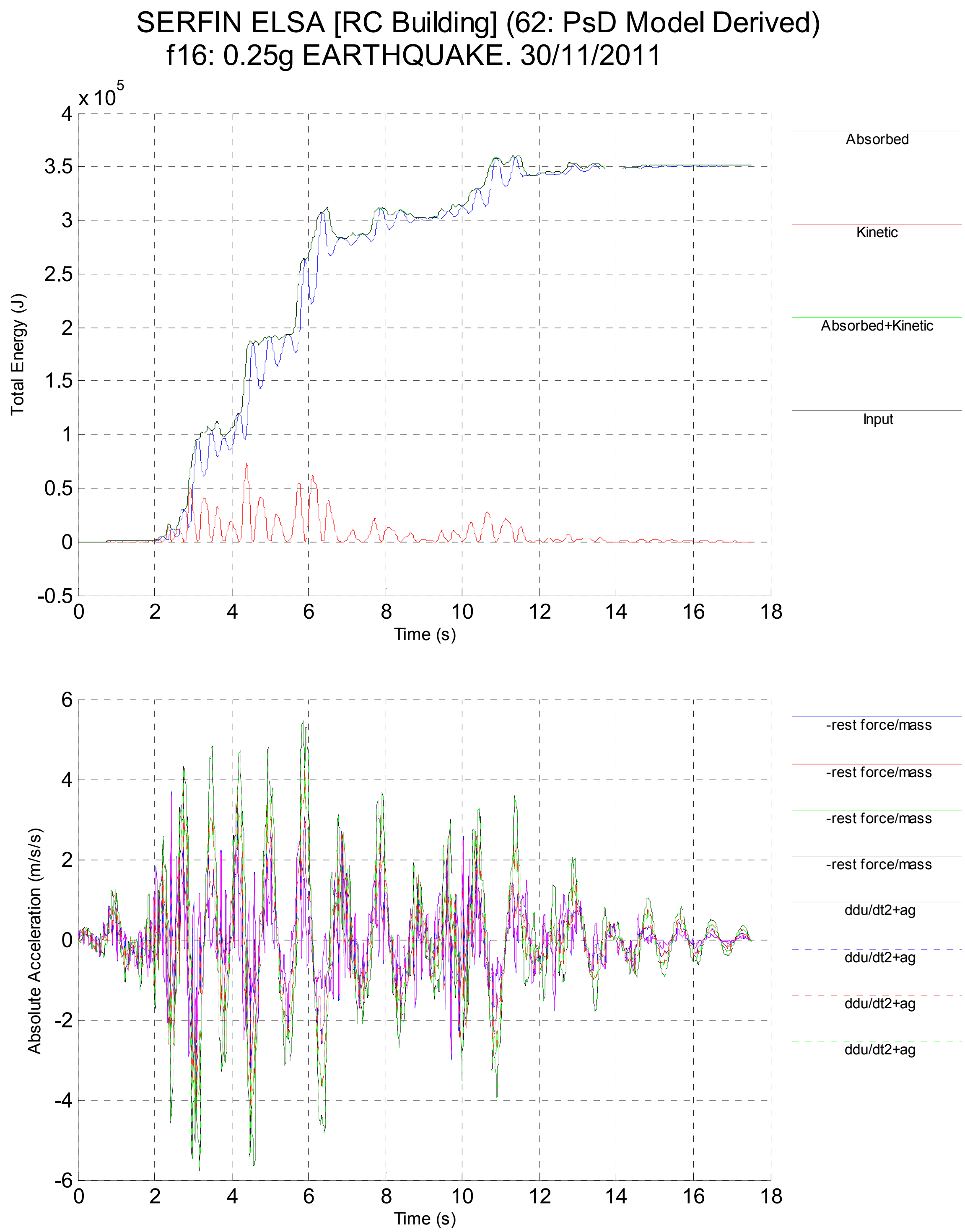

Figure 10.31 0.25g Test - Algorithm Energy and Acc. Histories

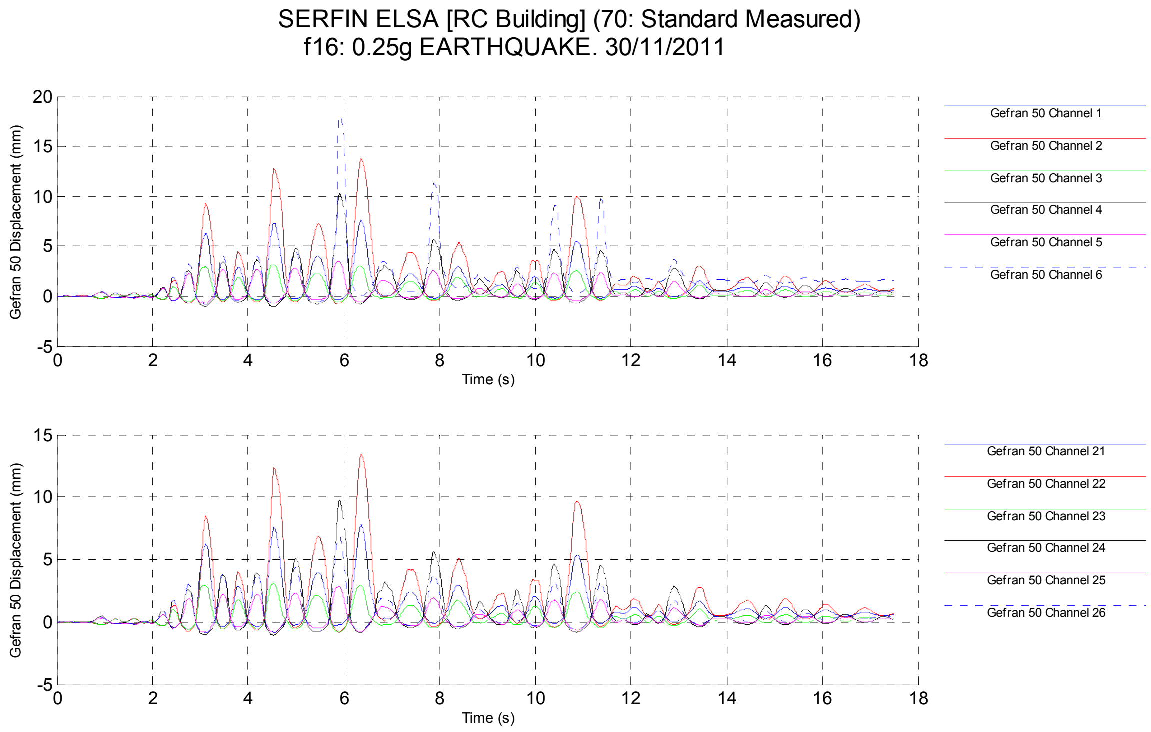

Figure 10.32 0.25g Test - Local Displacements - Ground Floor

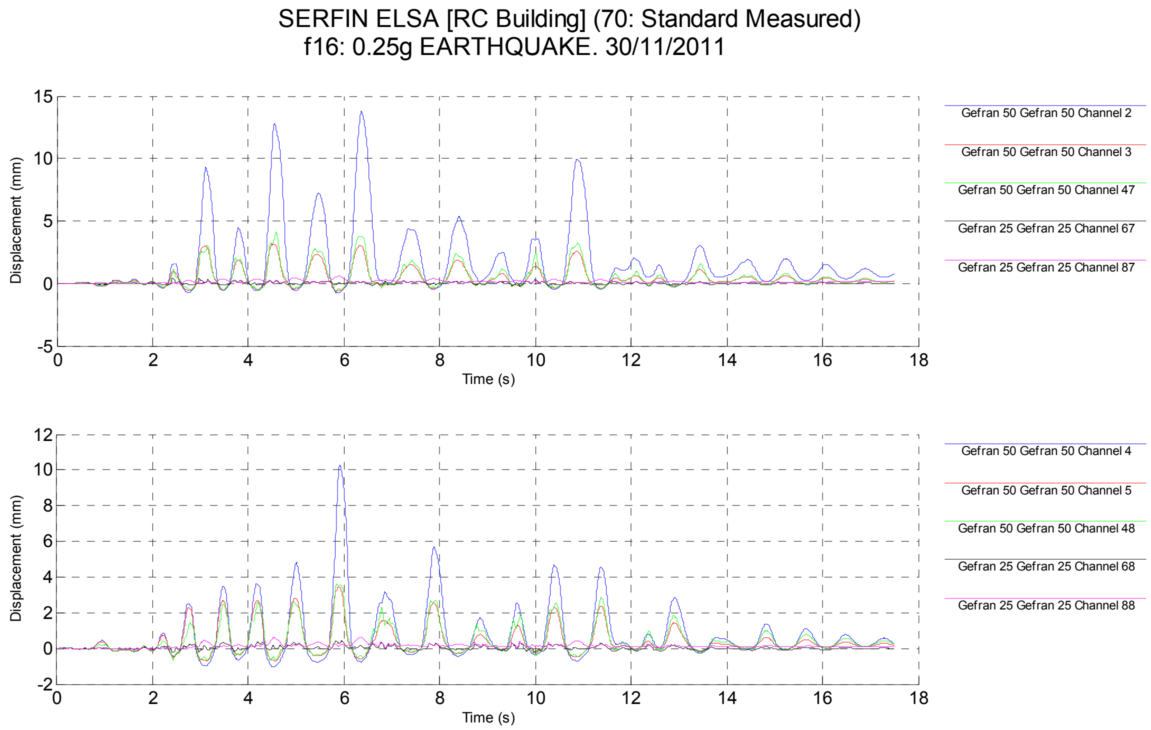

Figure 10.33 0.25g Test - Local Displacements - South Wall Columns

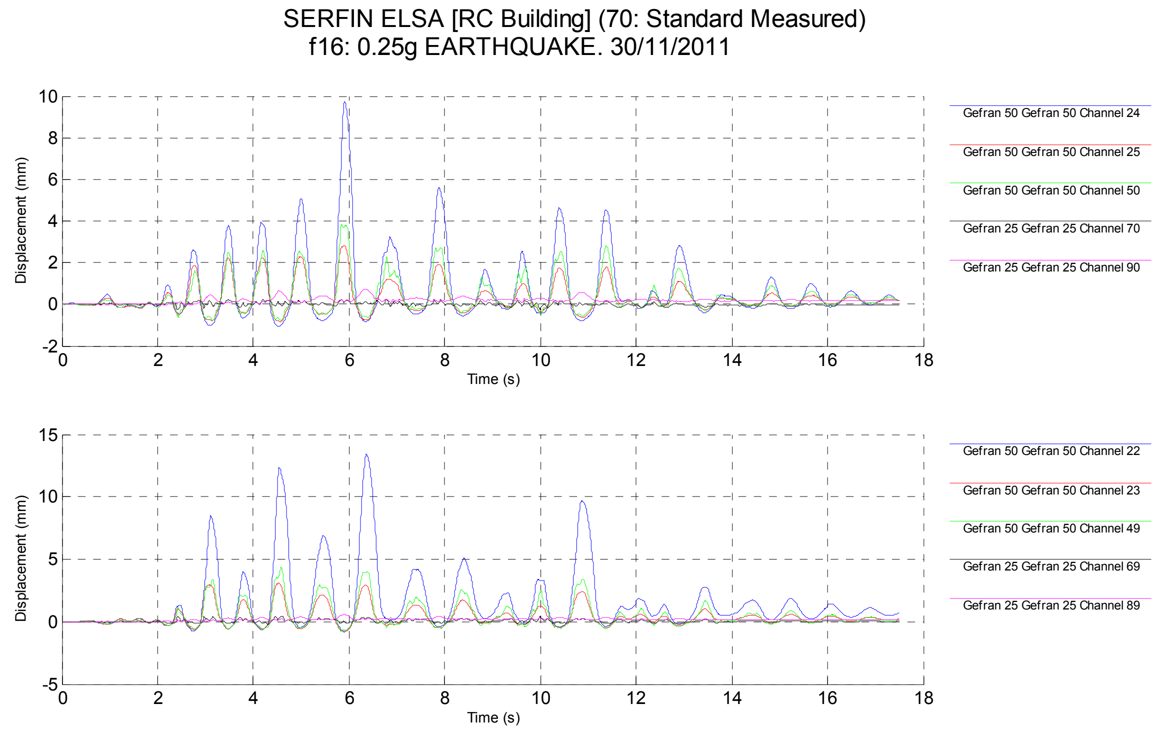

Figure 10.34 0.25g Test - Local Displacements - North Wall Columns

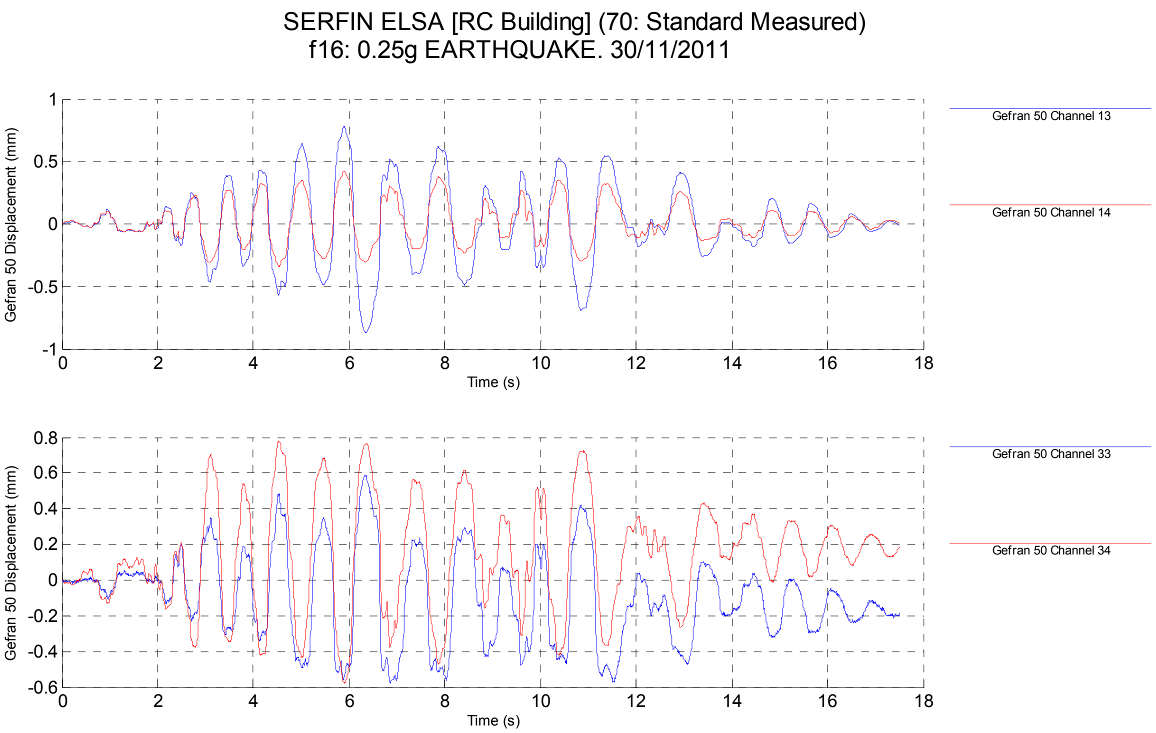

Figure 10.35 0.25g Test - Local Slip Displacements - Ground Beam

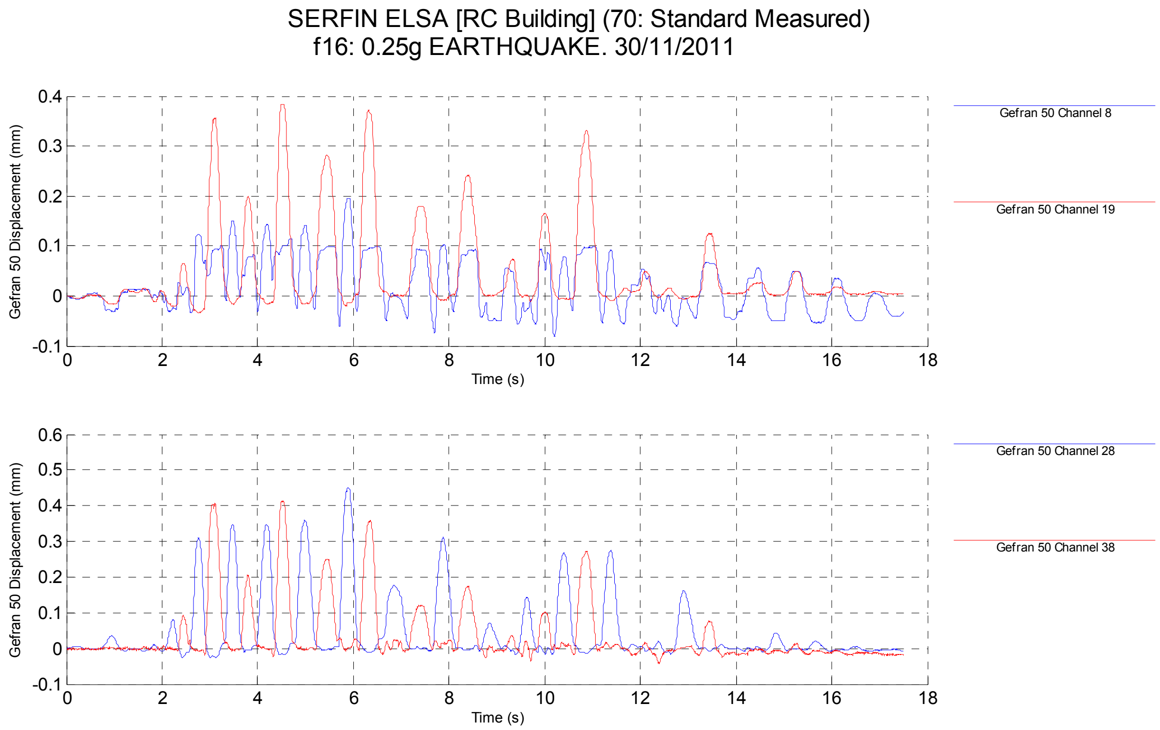

Figure 10.36 0.25g Test - Local Slip Displacements - Ground Column

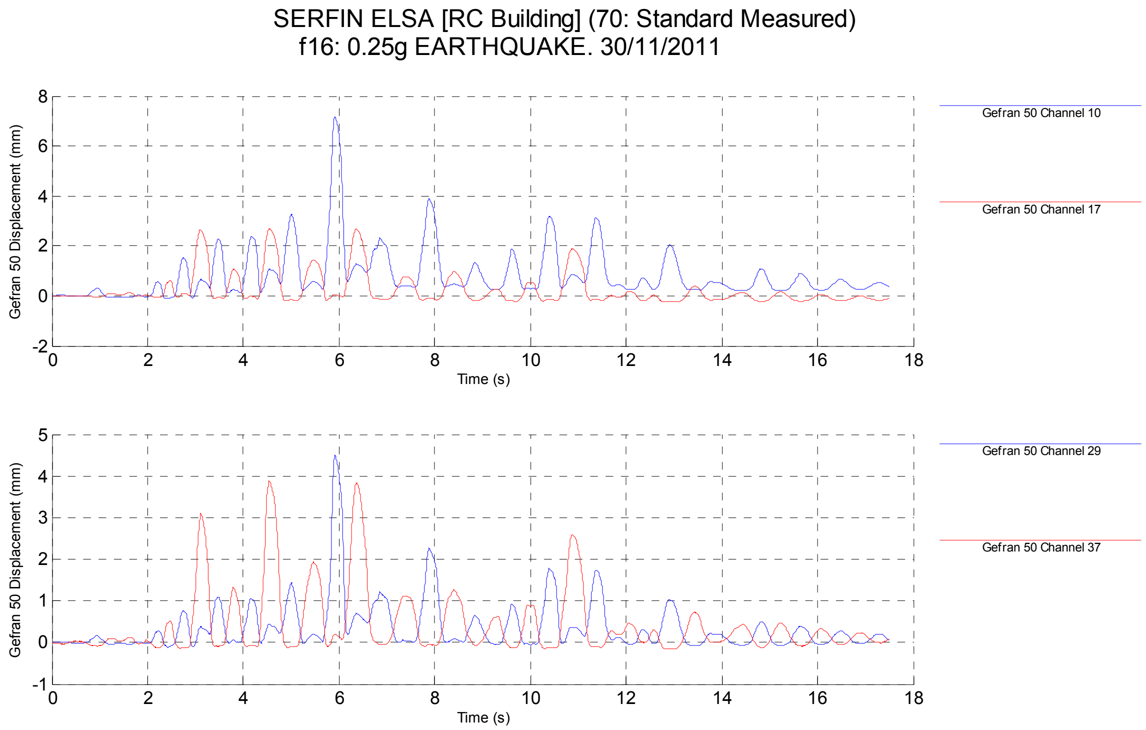

Figure 10.37 0.25g Test - Local Bottom Opening of the Wall, Ground Floor

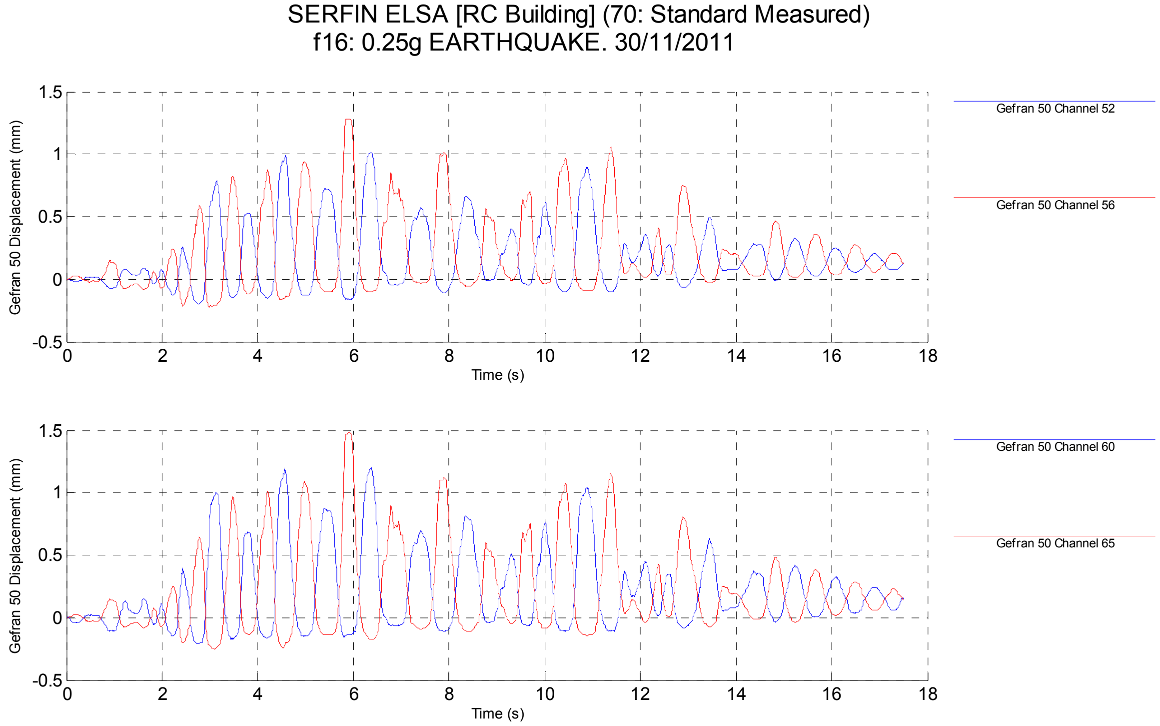

Figure 10.38 0.25g Test - Local Bottom Opening of the Wall, 1st Floor

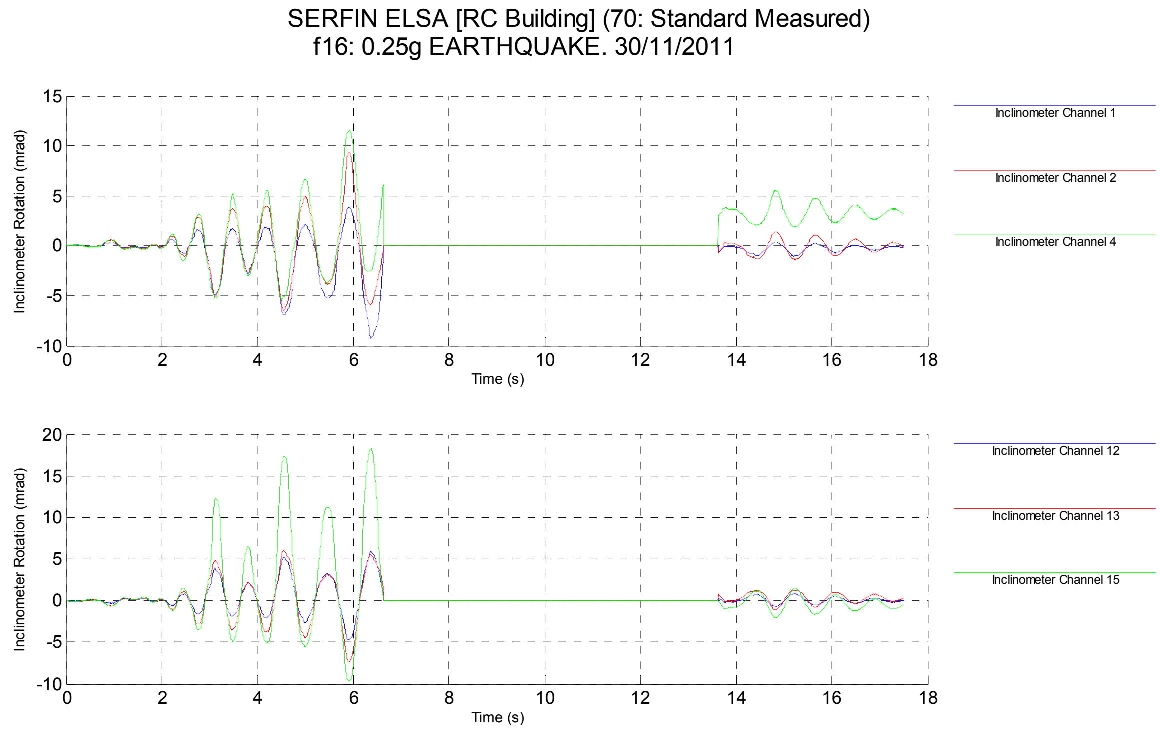

Figure 10.39 0.25g Test - Rotations - West Column

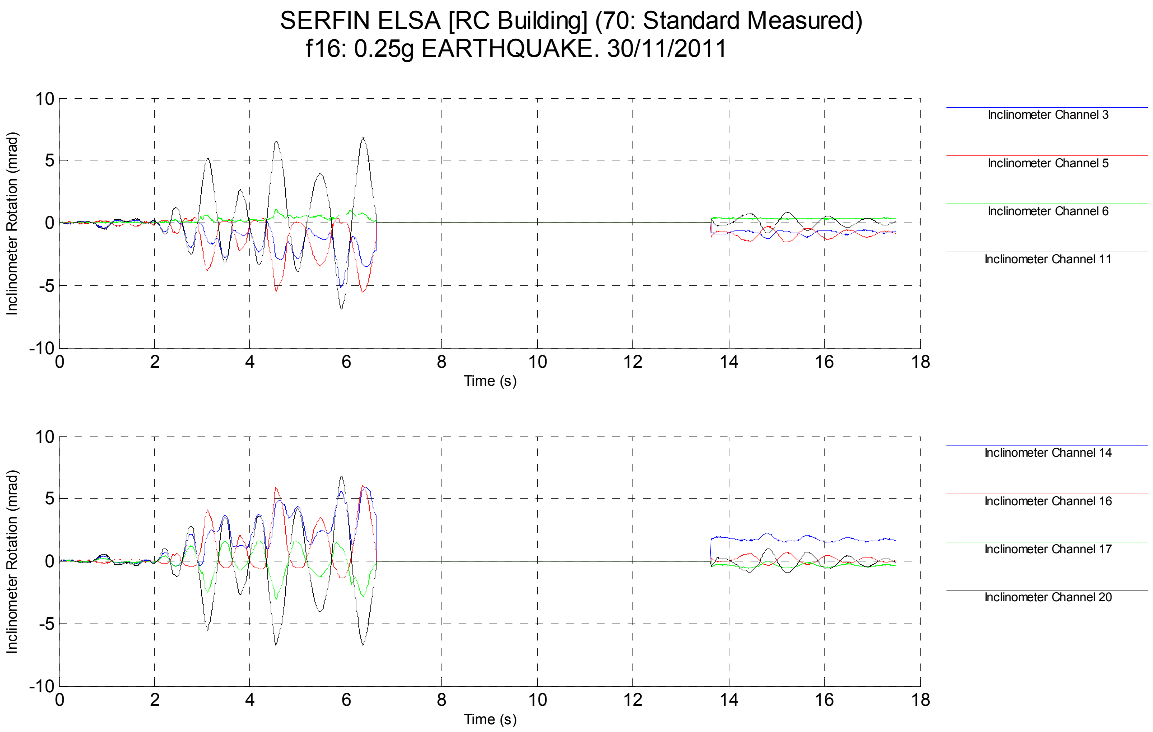

Figure 10.40 0.25g Test - Rotations - 1st Slab

CYCLIC TEST

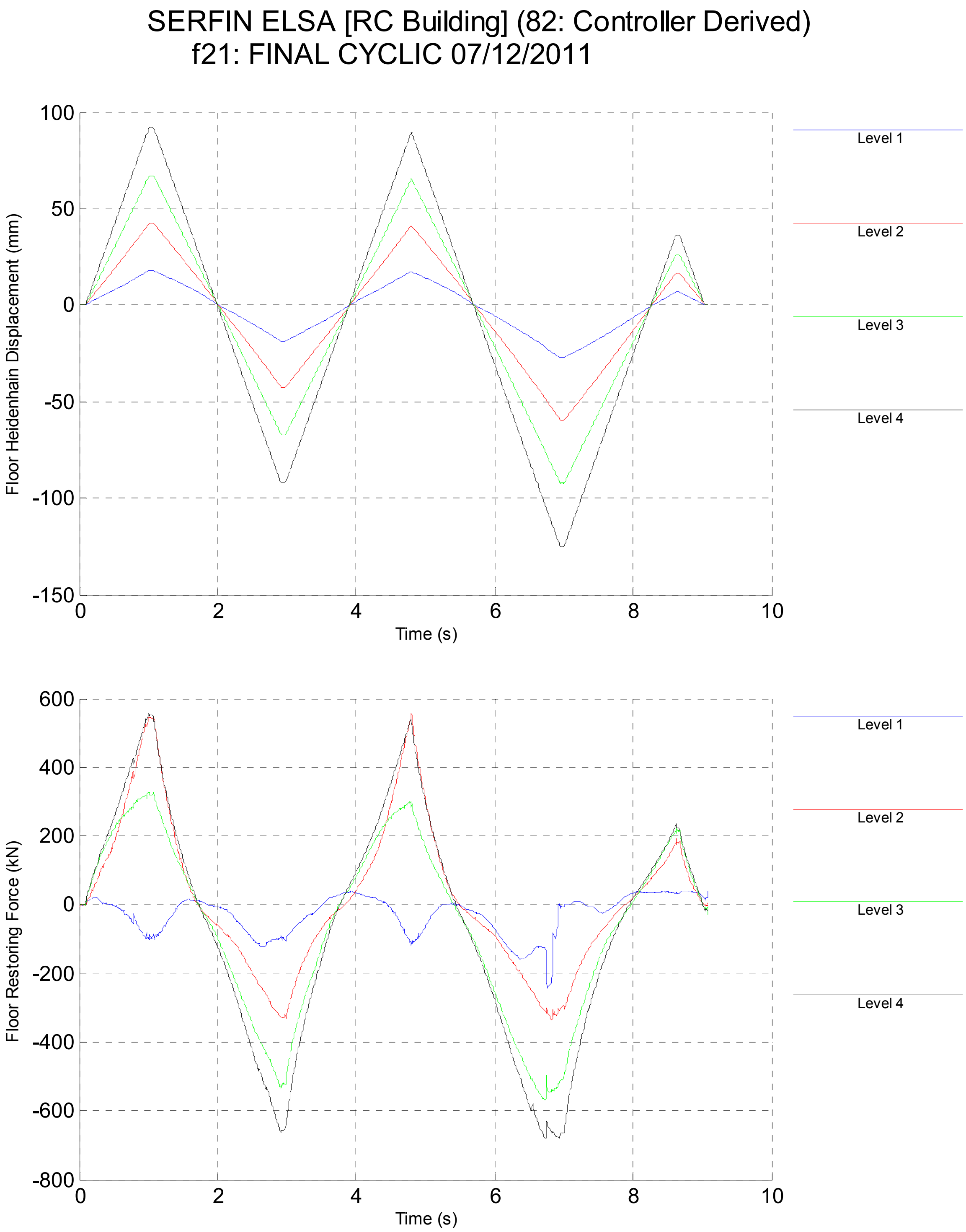

Figure 10.41 Cyclic Test - Floor Displacement and Force Histories

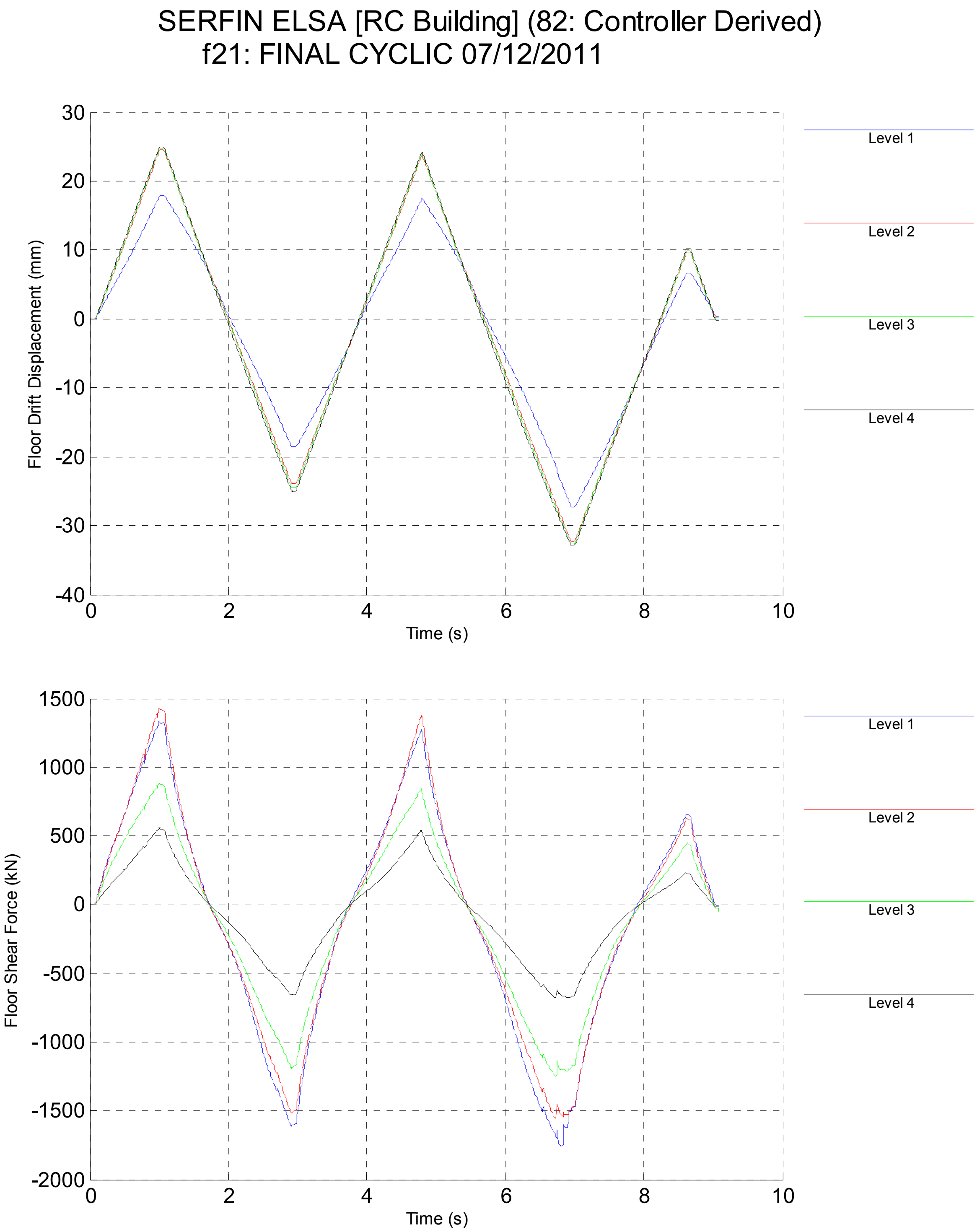

Figure 10.42 Cyclic Test - Floor Drift & Shear Force Histories

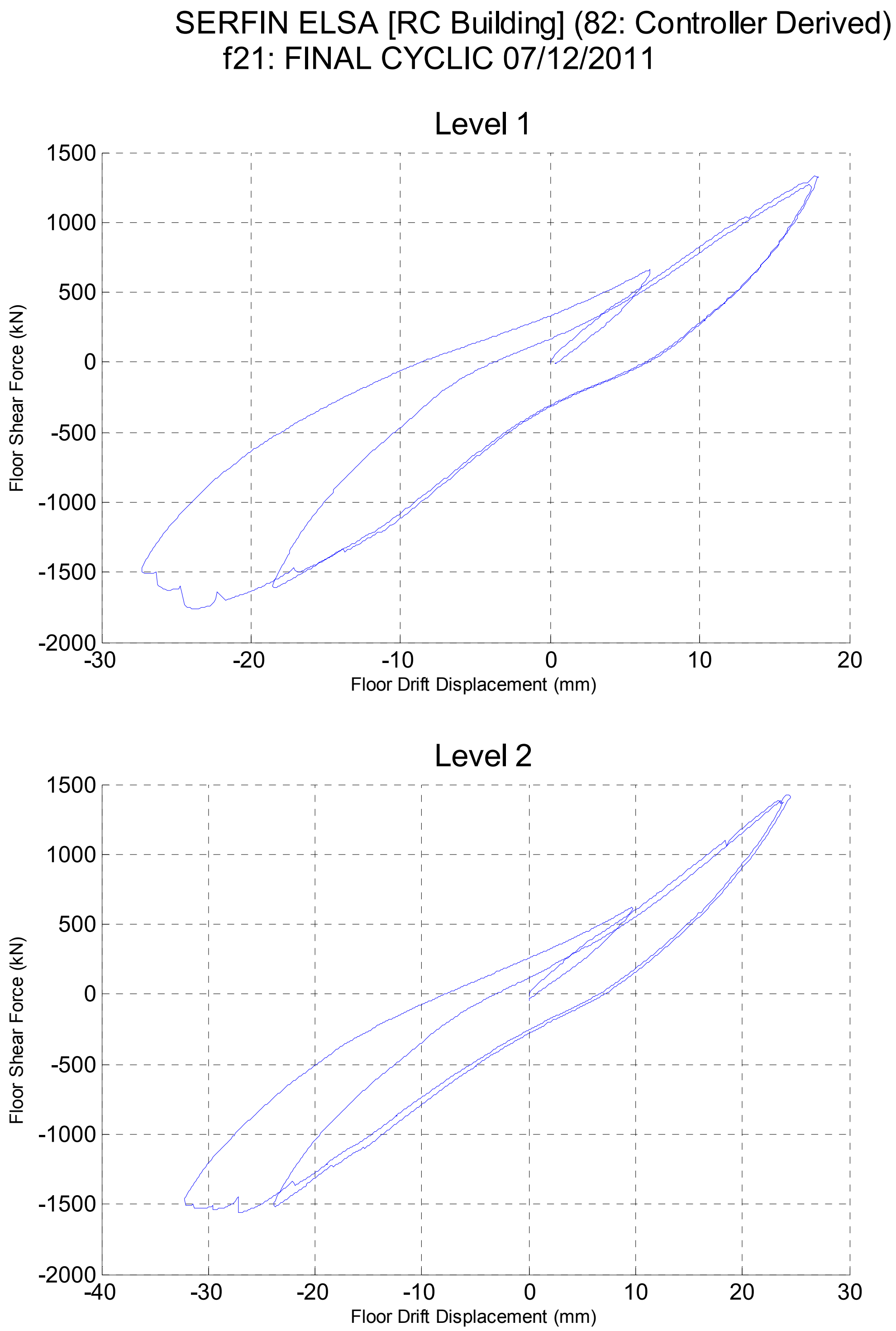

Figure 10.43 Cyclic Test - Floor Shear Force/Drift Cycle

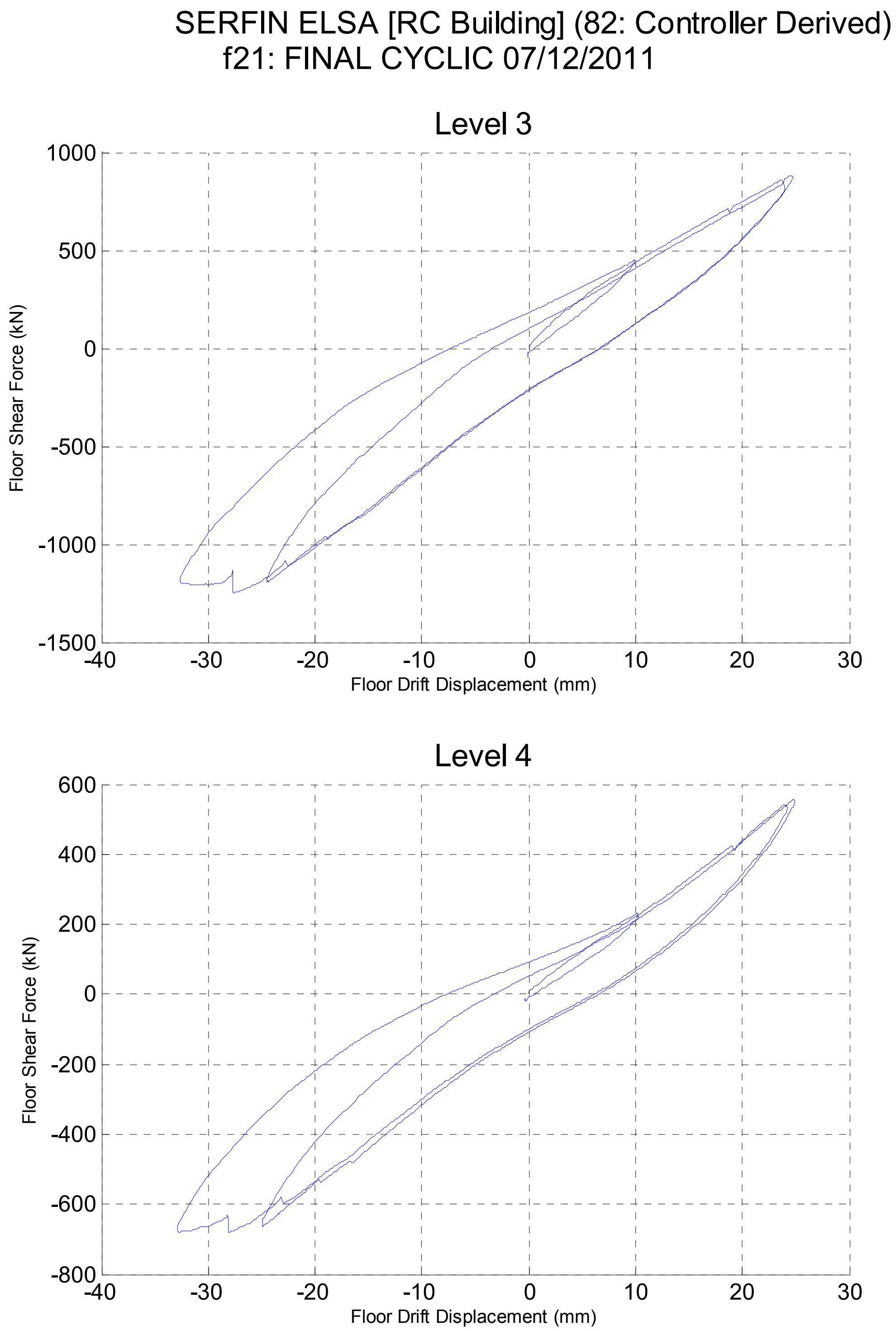

Figure 10.44 Cyclic Test - Floor Shear Force/Drift Cycle

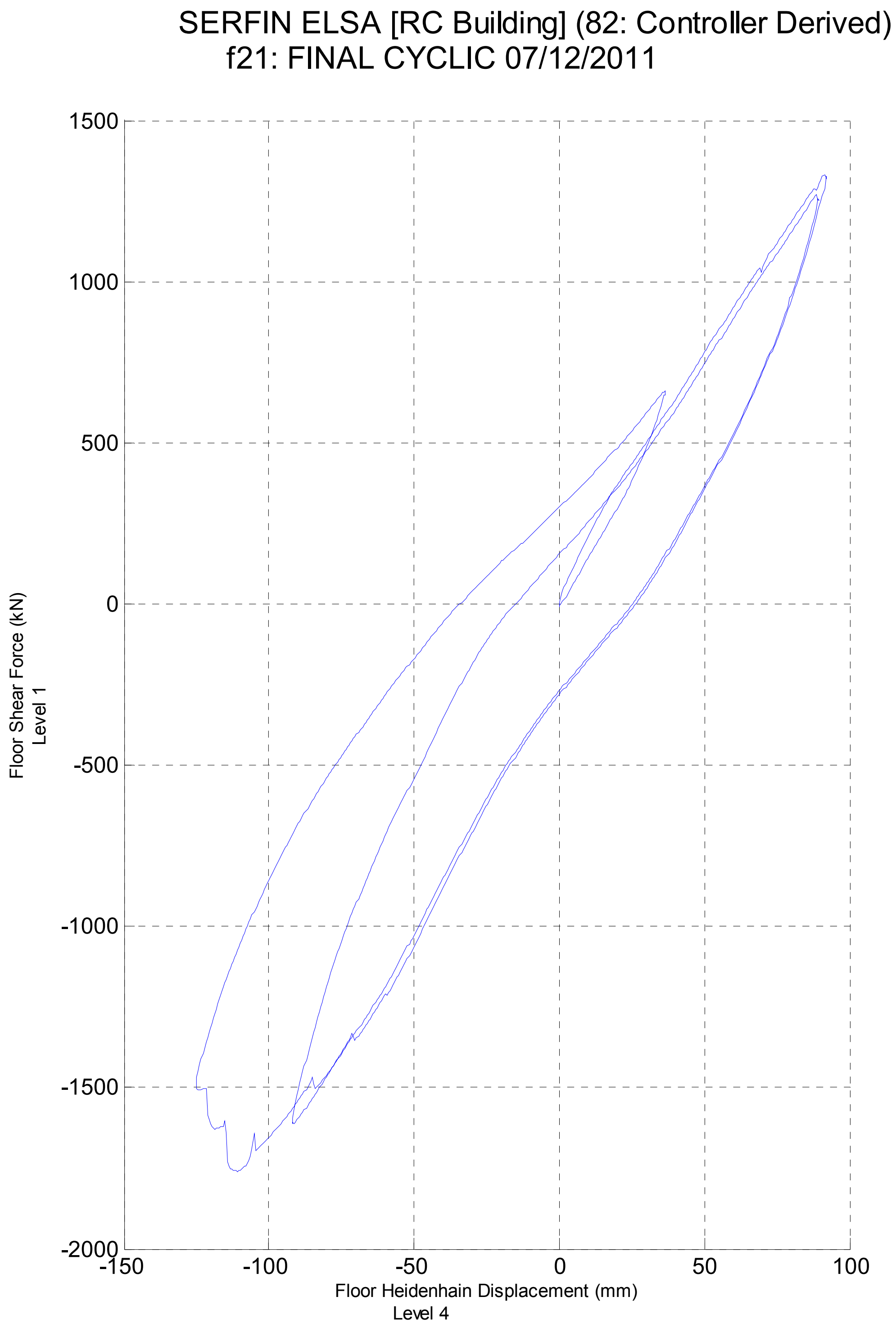

Figure 10.45 Cyclic Test - Base Shear Force/Top Displacement Cycle

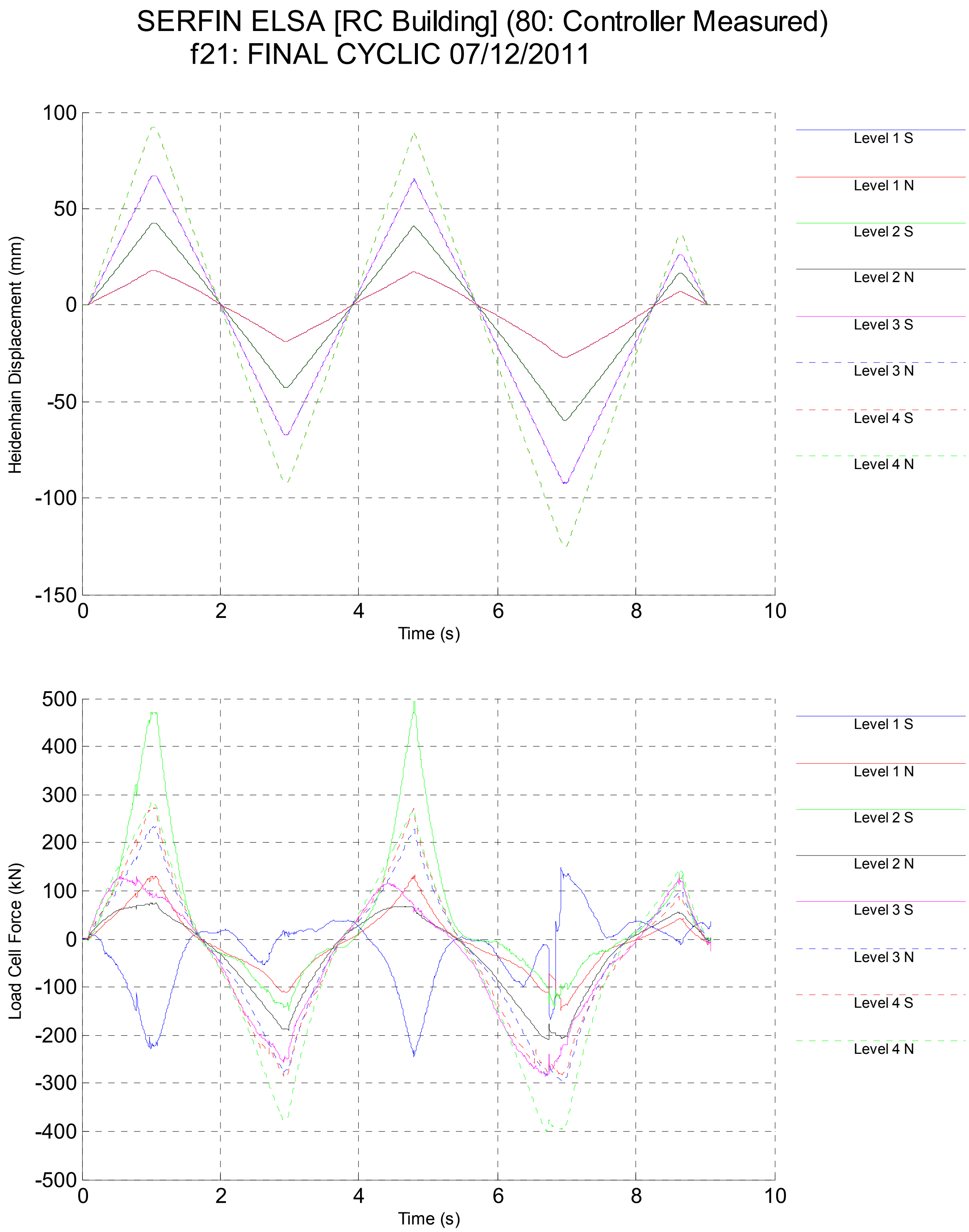

Figure 10.46 Cyclic Test - Controller Force & Displacement Histories

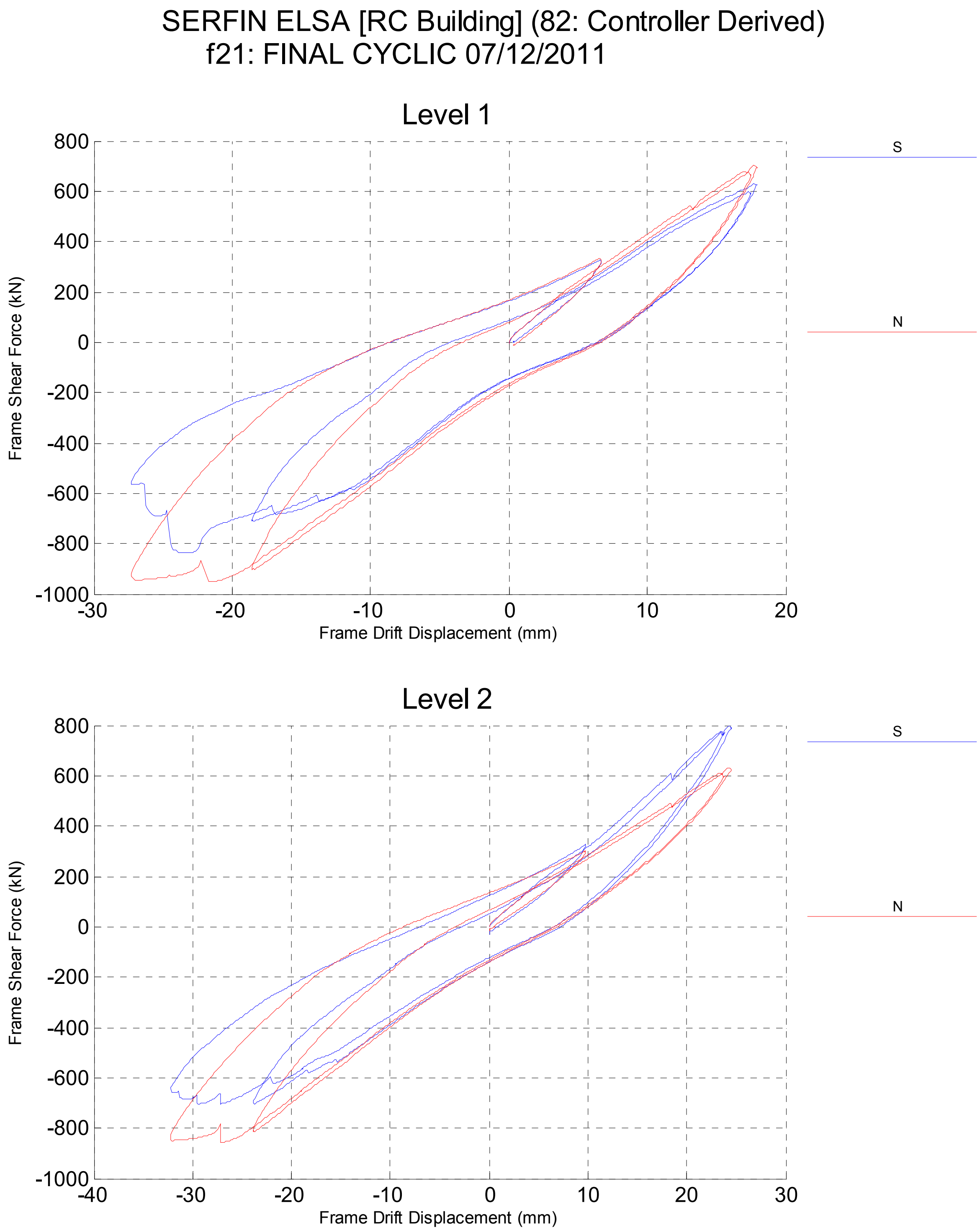

Figure 10.47 Cyclic Test - Frame Shear Force/Drift Cycle

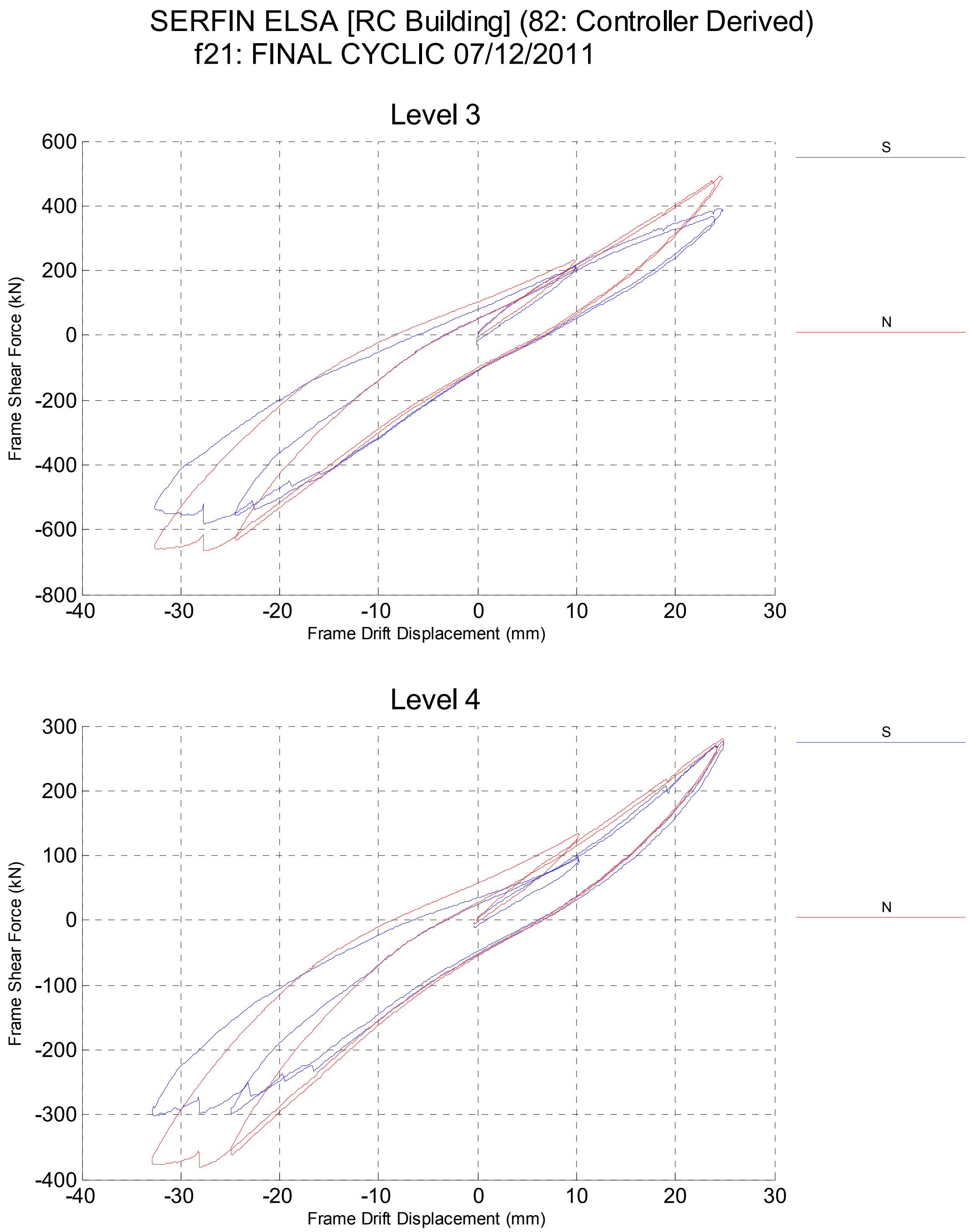

Figure 10.48 Cyclic Test - Frame Shear Force/Drift Cycle

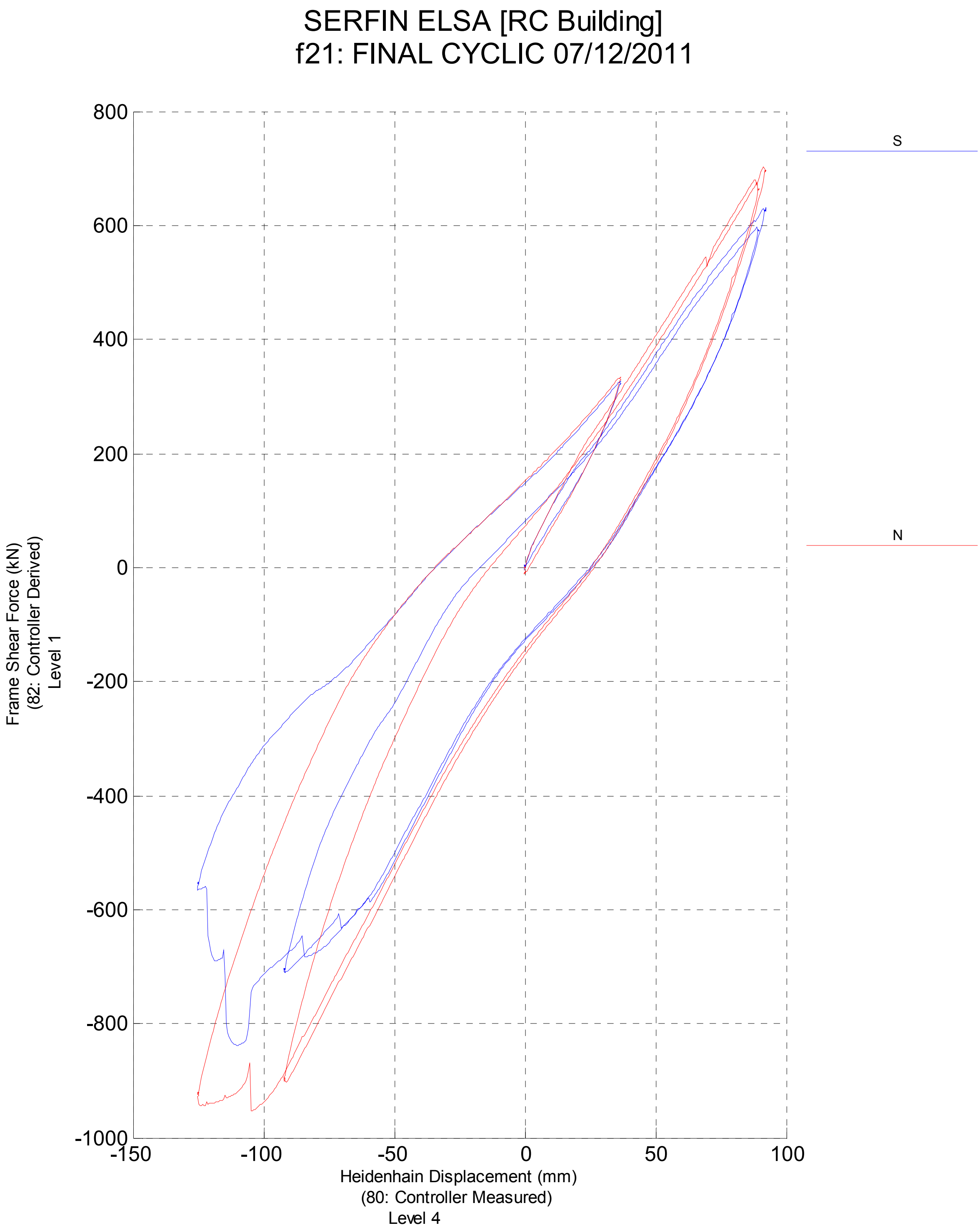

Figure 10.49 Cyclic Test - Frame Base Shear/Top Displacement

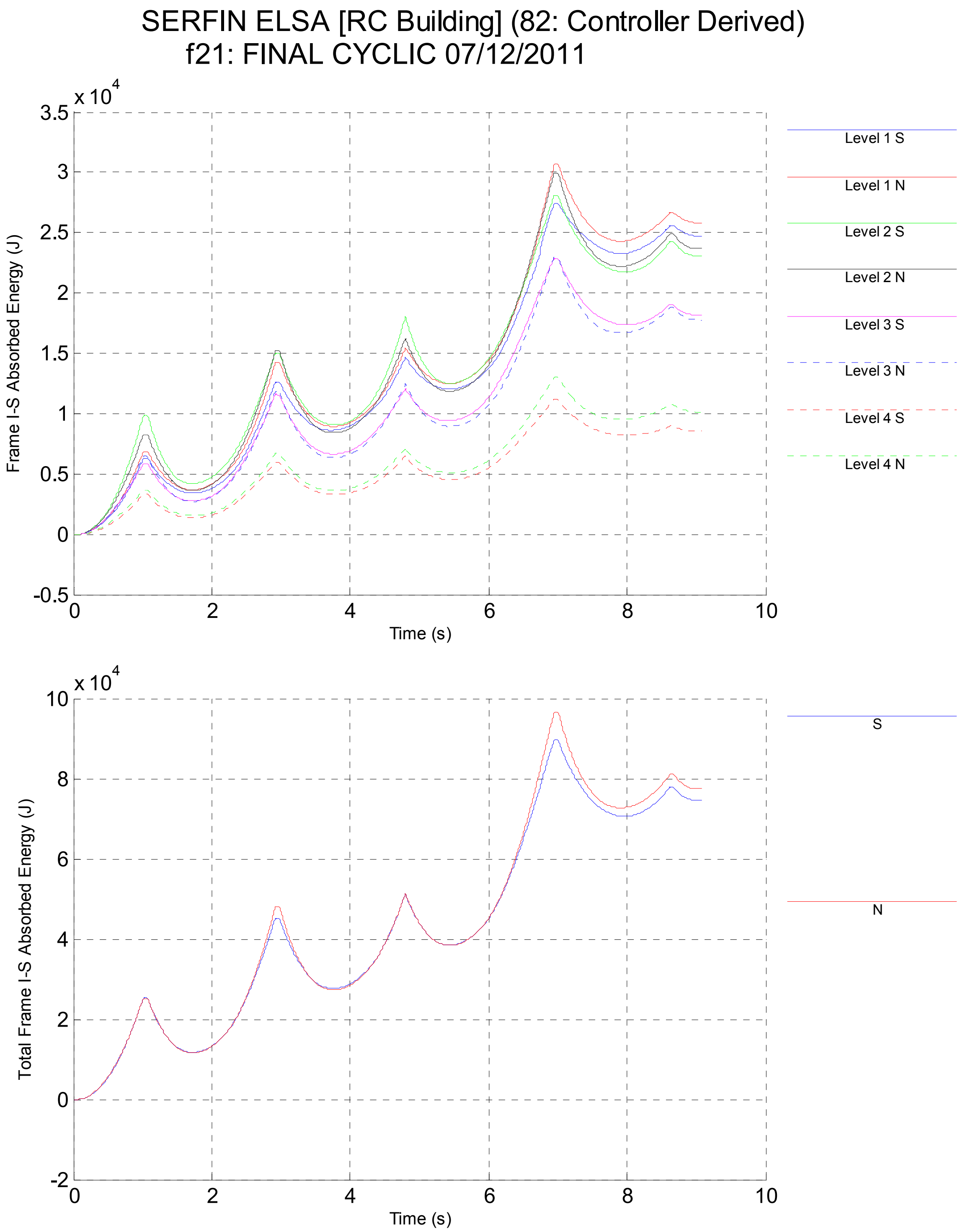

Figure 10.50 Cyclic Test - Frame Energy Histories

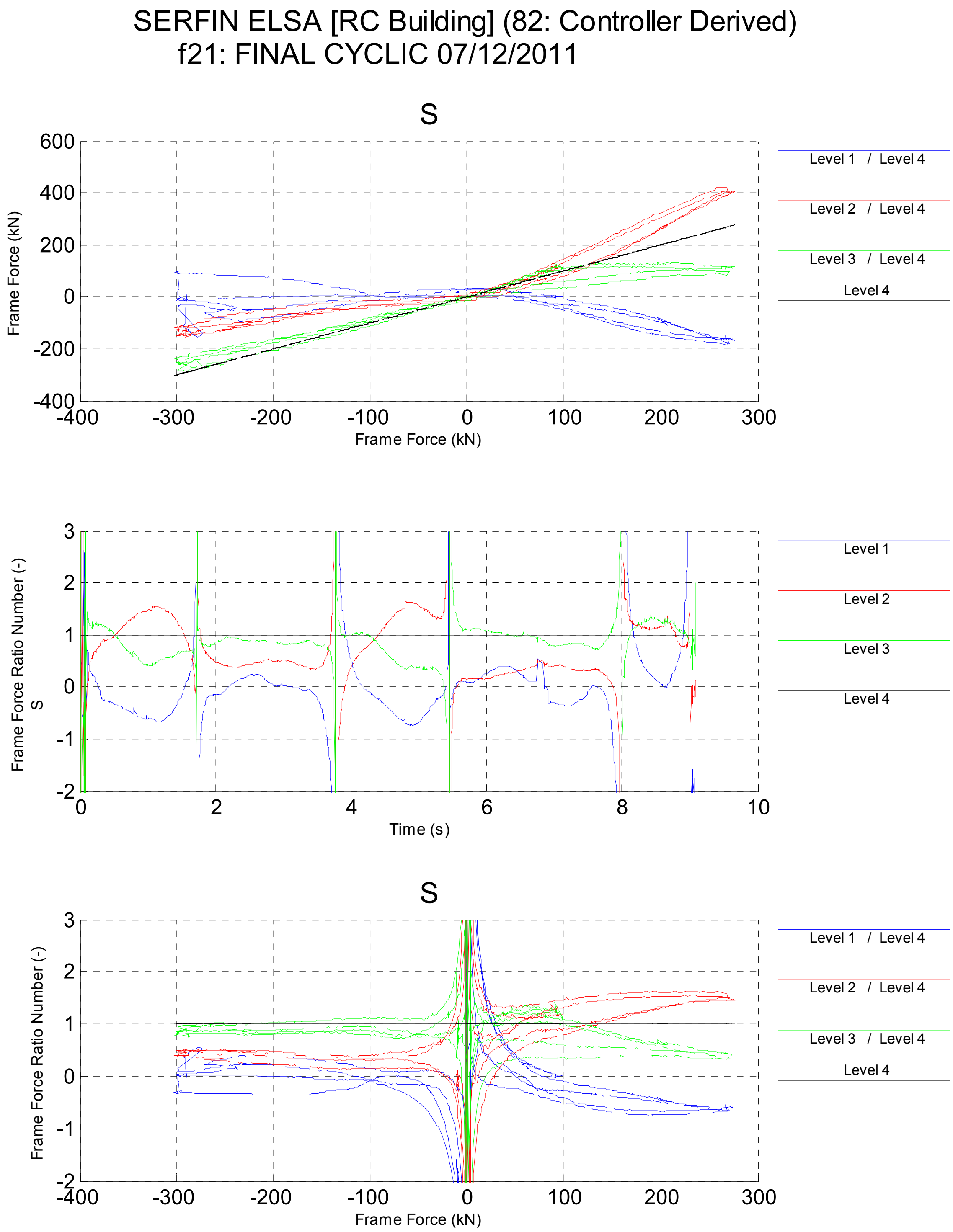

Figure 10.51 Cyclic Test - South Frame Force Ratio Histories

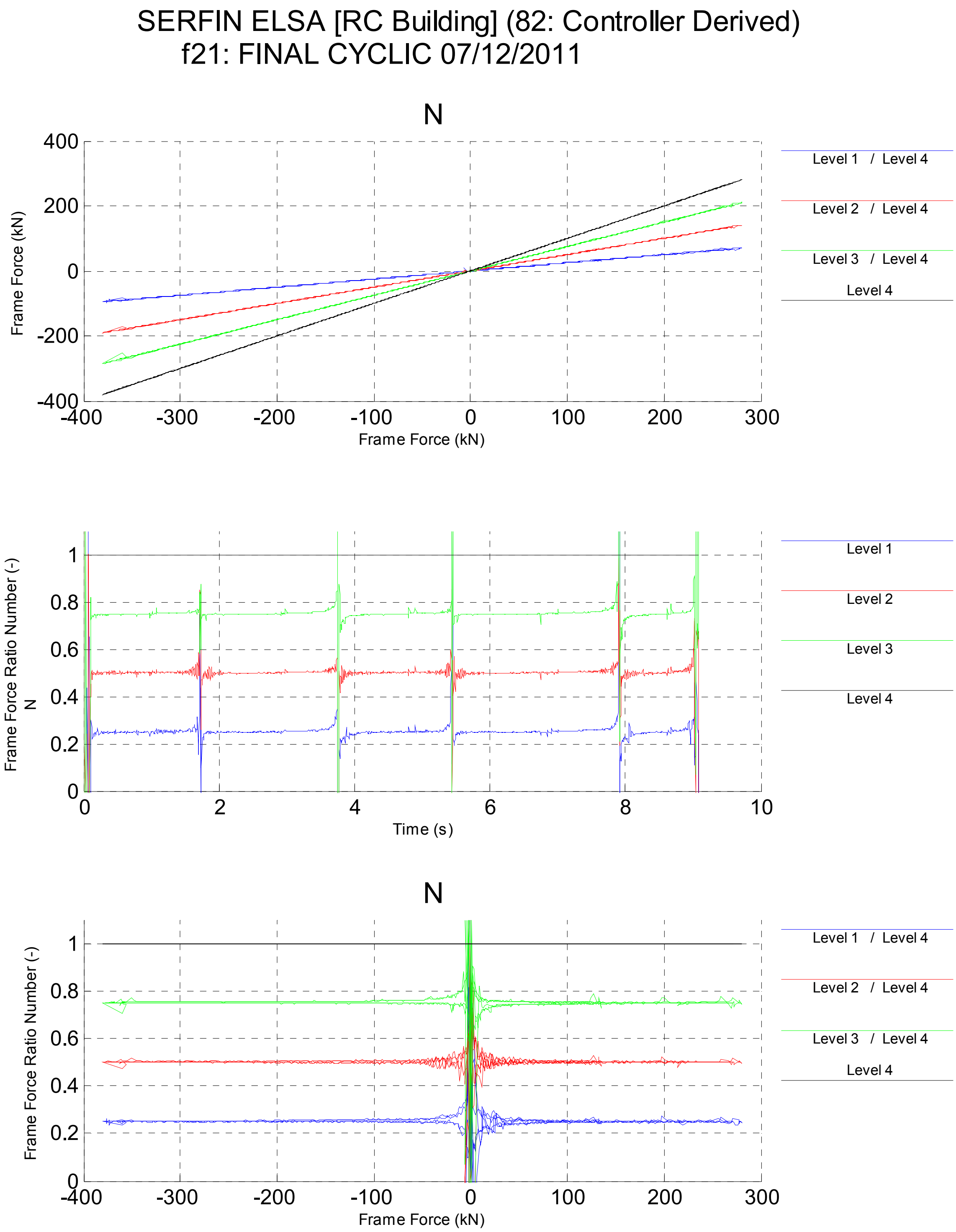

Figure 10.52 Cyclic Test - North Frame Force Ratio Histories

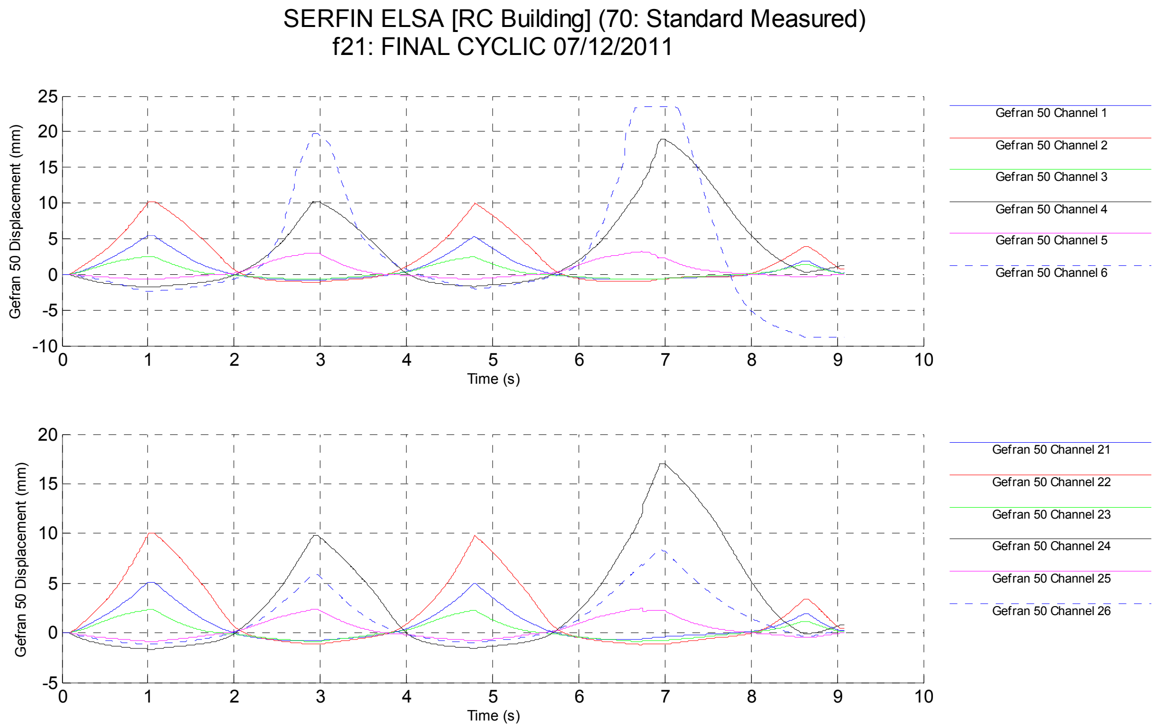

Figure 10.53 Cyclic Test - Local Displacements - Ground Floor

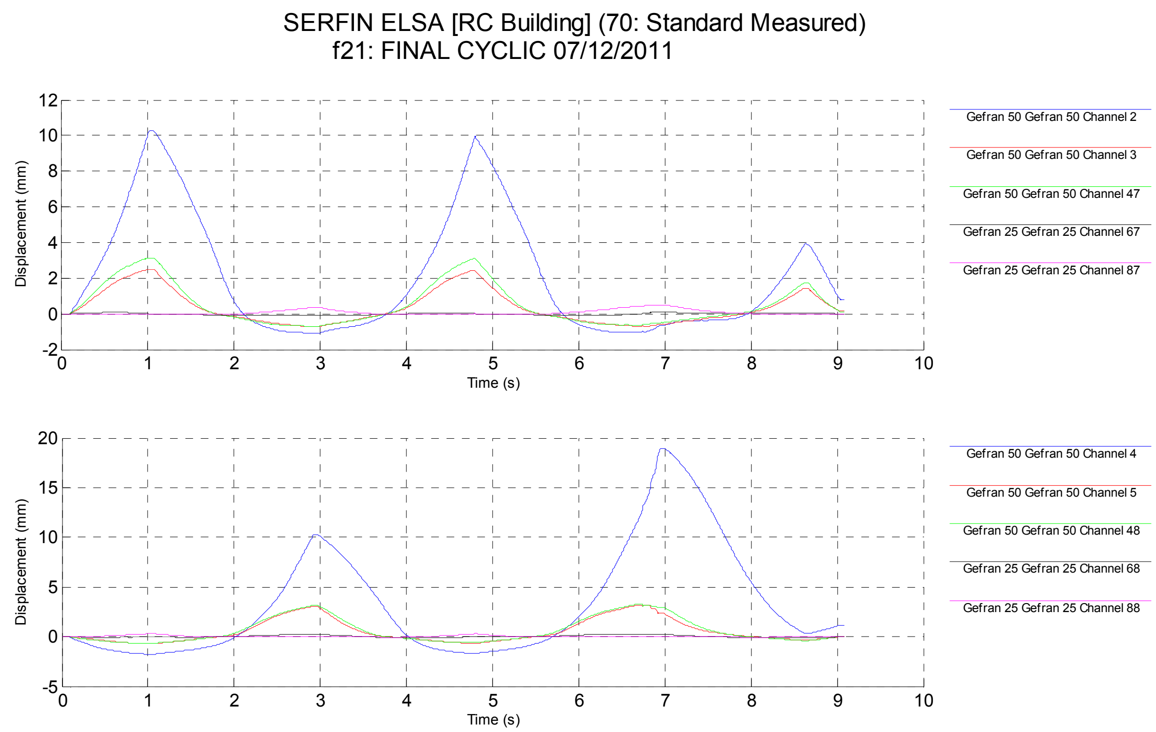

Figure 10.54 Cyclic Test - Local Displacements - South Wall Columns

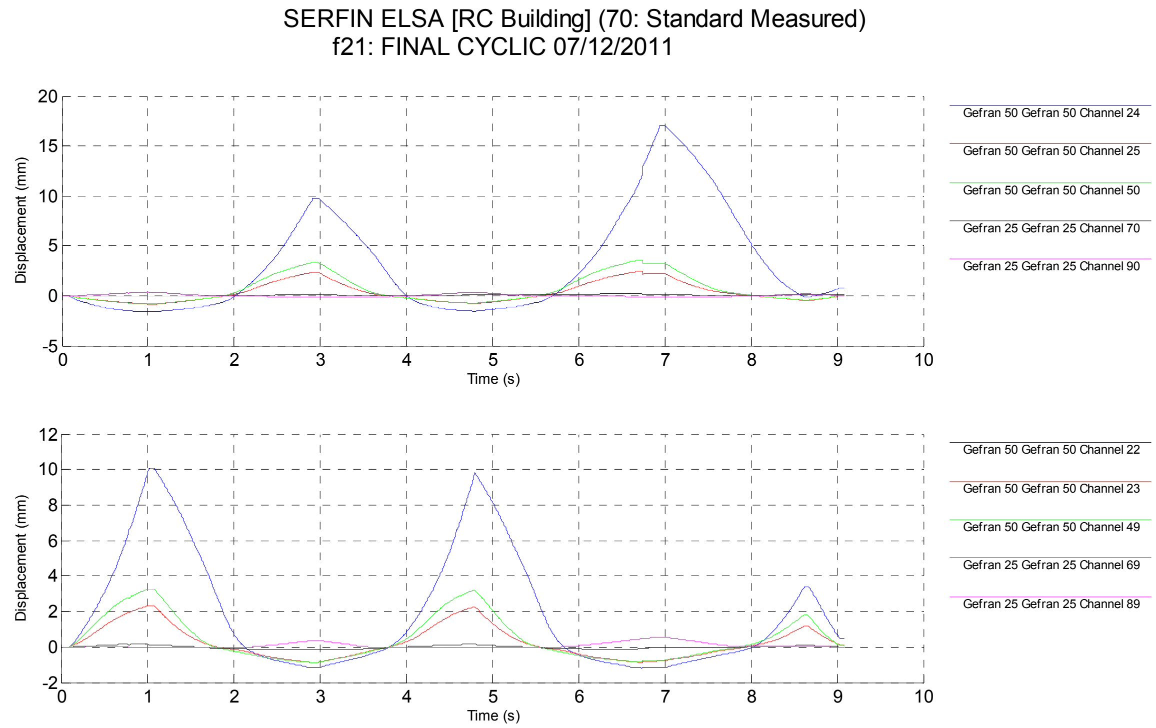

Figure 10.55 Cyclic Test - Local Displacements - North Wall Columns

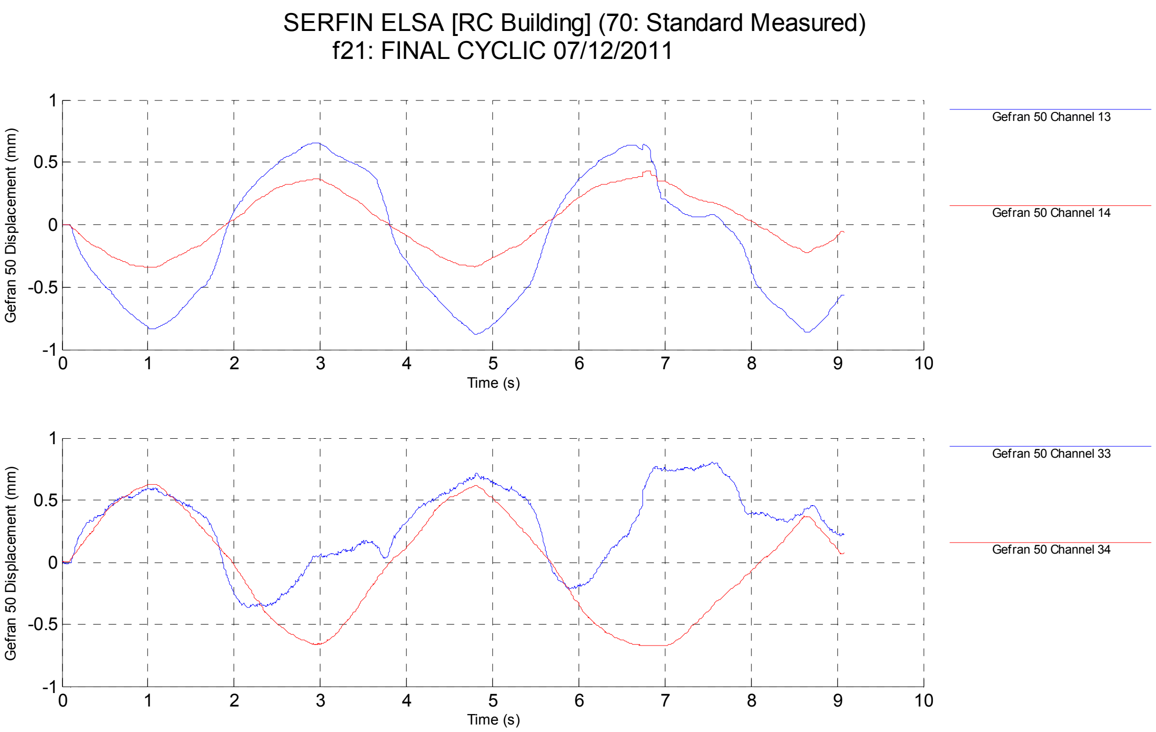

Figure 10.56 Cyclic Test - Local Slip Displacements - Ground Beam

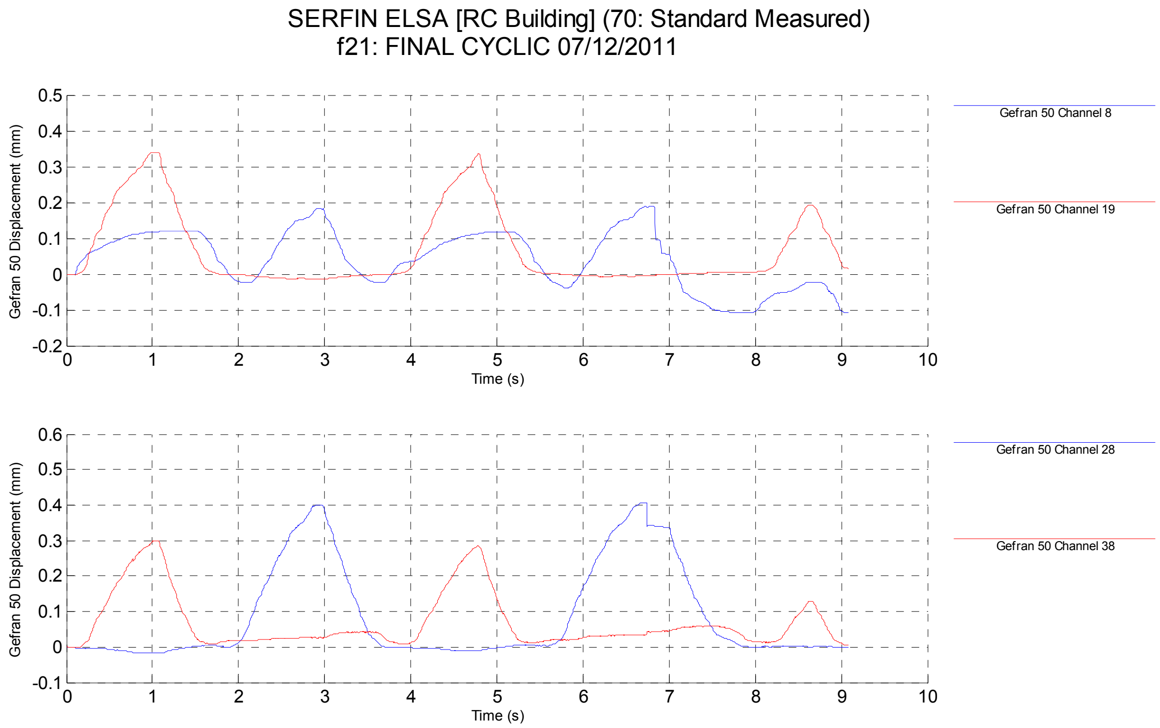

Figure 10.57 Cyclic Test - Local Slip Displacements - Ground Column

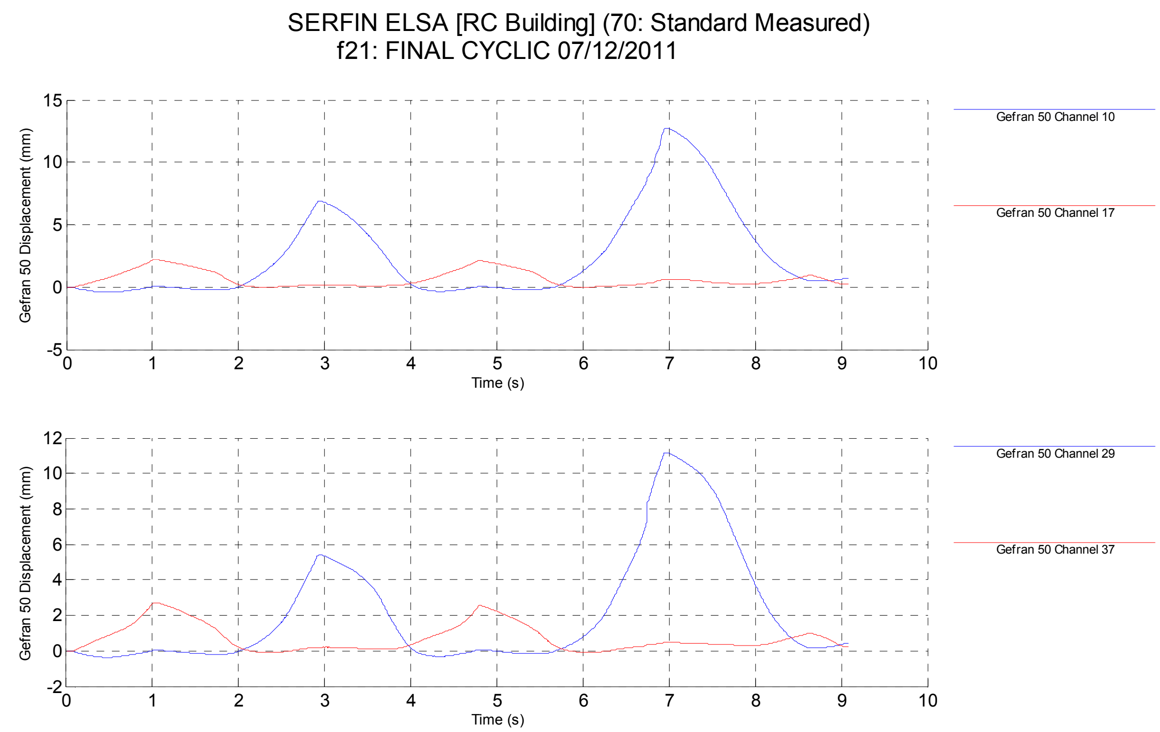

Figure 10.58 Cyclic Test - Local Bottom Opening of the Wall, Ground Floor

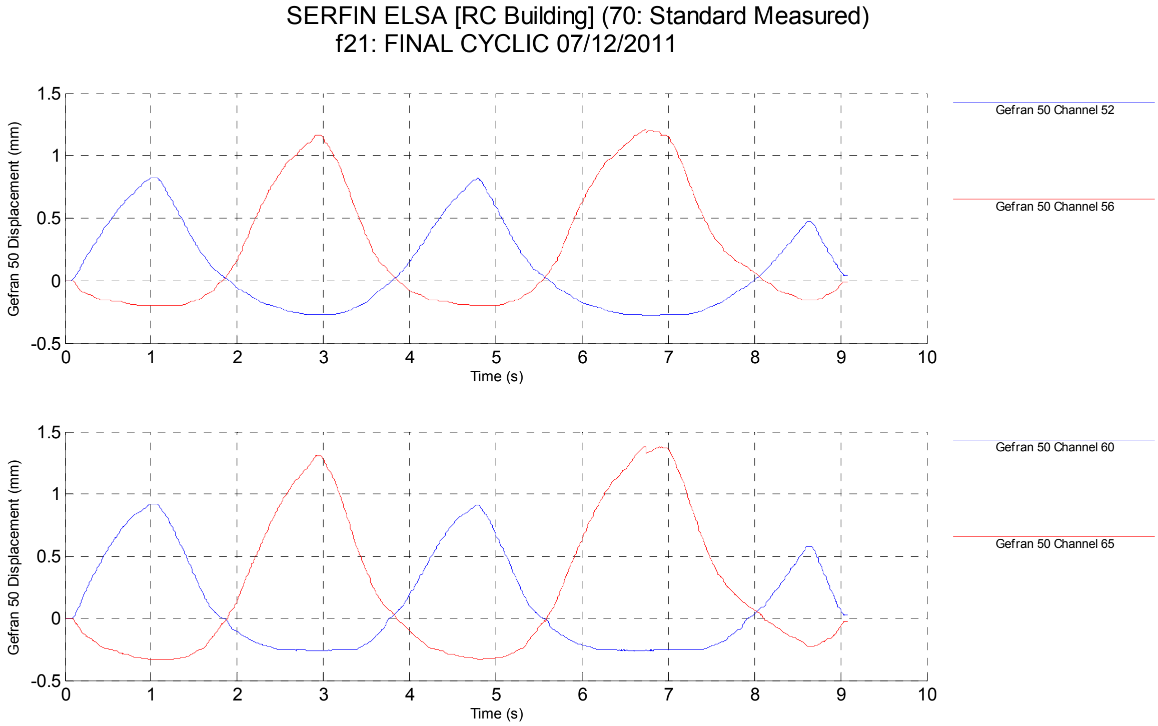

Figure 10.59 Cyclic Test - Local Bottom Opening of the Wall, 1st Floor

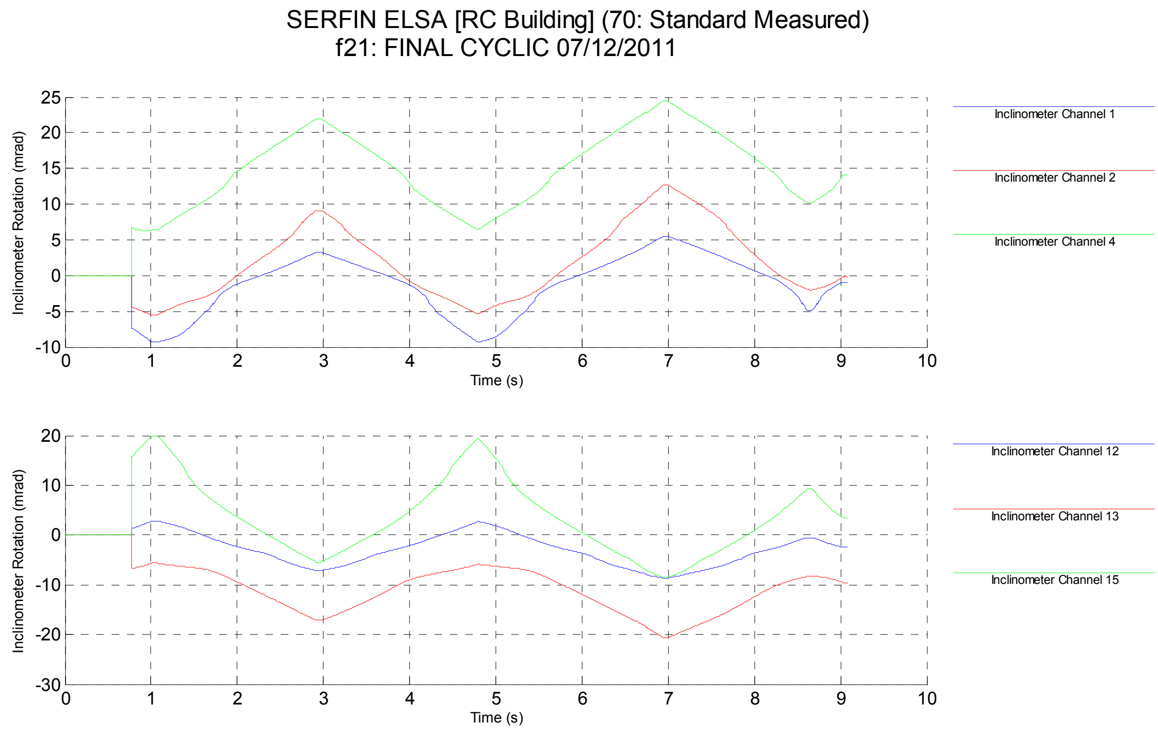

Figure 10.60 Cyclic Test - Rotations - West Column

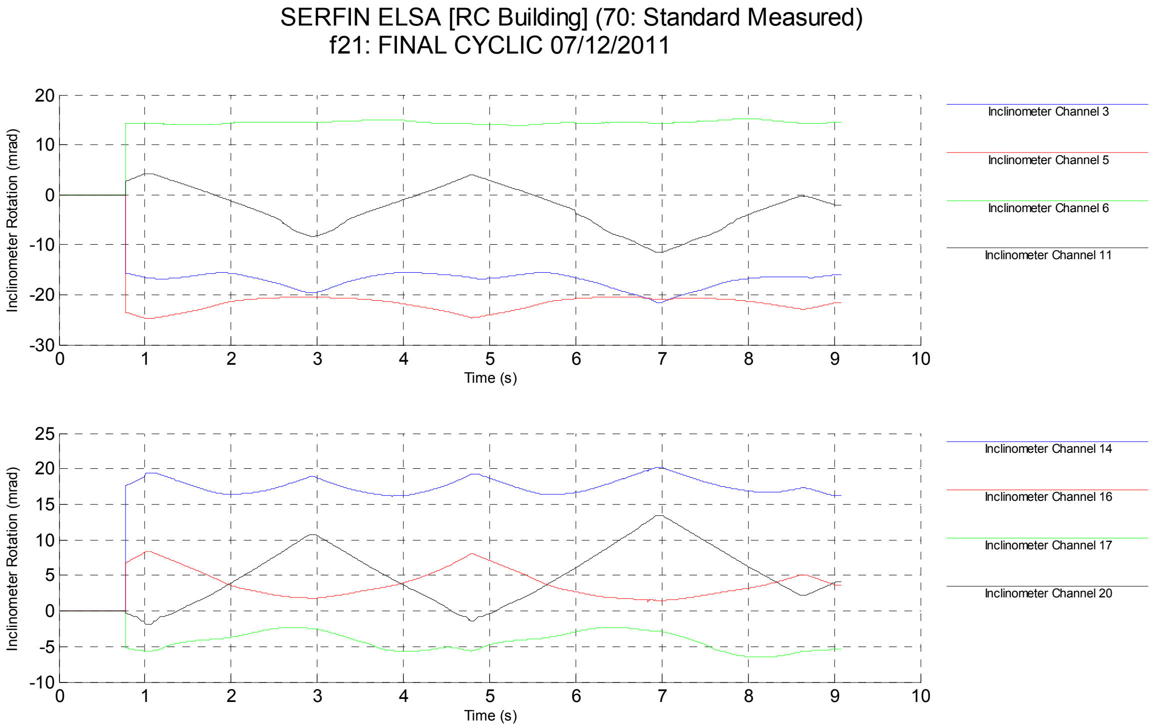

Figure 10.61 Cyclic Test - Rotations - 1st Slab

European Commission
EUR 26470 EN– Joint Research Centre – Institute for the Protection and Security of the Citizen

Title: **Seismic Retrofitting of RC Frames with RC Infilling (SERFIN Project)**

Author(s): Martin Poljanšek, Fabio Taucer, Javier Molina Ruiz, Christis Chrysostomou, Nicholas Kyriakides, Toula Onoufriou,Panayiotis Roussis, Panagiotis Kotronis, Telemachos Panagiotakos, Antonis Kosmopoulos

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Abstract

The effectiveness of seismic retrofitting of multi-storey multi-bay RC-frame buildings by converting selected bays into new walls through infilling with reinforced concrete (RC) was studied experimentally at the ELSA facility of the Joint Research Centre in Ispra (Italy). A full-scale model was tested with the pseudo-dynamic method and consisted of two four-storey (12m tall) three-bay (8.5m long) parallel frames linked through 0.15m slabs with the central bay (2.5m) infilled with a RC wall. The frames were designed and detailed for gravity loads only and are typical of similar frames built in Cyprus in the 1970's.

Different connection details and reinforcement percentages for the two infilled frames were used in order to study their effects in determining structural response. The results of the pseudo-dynamic and cyclic tests performed on the specimen with the new walls show five times higher resistance to earthquake loads when compared to typical building construction in Cyprus in the 1970's.

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