RESEARCH REPORT

VTT-R-00987-11

Details of retrofit solution using LGS shear walls and column bracketing

Authors: Matti Mielonen, Ludovic Fülöp

Confidentiality: Confidential
Public after closing the project (July 2010).

Report's title	
Details of retrofit solution using LGS shear walls and column bracketing	
Customer, contact person, address Commission of the European Communities	Order reference RFS-PR-06054
Project name STEELRETRO	Project number/Short name 12597/STEELRETRO
Author(s) Matti Mielonen, Ludovic Fülöp	Pages 38
Keywords r.c. structure, seismic retrofit, light-gauge steel shear walls	Report identification code VTT-R-00987-11

Summary

This report is part of the project STEELRETRO, which aim was to develop seismic retrofitting solutions for a reinforced concrete building. This work is based on earlier study: "*LGS steel shear wall system to retrofit or upgrade r.c. frame structures*" (VTT-R-04770-09).

The light gauge steel (LGS) shear wall solutions presented in the previous report were investigated further and reliability of SeismoStruct modelling (finite element pushover analysis) was compared against available literature studies on concrete columns, steel plate shear walls and steel jacketing. The usability of the results was improved by providing principles of connection detailing for manufacturing.

The proposed retrofit solution of using LGS shear walls on external frames that are anchored/bolted to the concrete frame should provide acceptable performance with minor rework of the existing structure and relatively easy installation procedure.

Public after closing the project (July 2010).

Confidentiality	Confidential

Espoo 31.1.2011

Written by	Reviewed by	Accepted by
Matti Mielonen Ludovic Fülöp Research Scientists	Hakola Ilkka Senior Research Scientist	Eila Lehmus Technology Manager

VTT's contact address
P.O. Box 1000, FI-02044 VTT, Finland

Distribution (customer and VTT)
{Customer, VTT and other distribution. In confidential reports the company, person and amount of copies must be named. Continue to next page when necessary.}

Preface

The report is part of the project STEELRETRO, Work Package 3. The overall aim of the STEELRETRO project is to *“set up steel solutions for the seismic retrofit of existing buildings, furnishing design and construction methodologies, tools for dimensioning of elements and connections as well as for cost estimation”*.

Specifically, Work Package 3 aims at analyzing and designing *“steel solutions to retrofit or upgrade vertical systems of existing reinforced concrete building, in terms of strength or stiffness, by means of steel concentric bracing systems, steel eccentric bracing systems or shear steel/composite walls.”* and *“steel solutions to retrofit or upgrade vertical systems of existing masonry building coupling the existing structure with new a steel structure or with a bracing systems.”*

WP 3 also aims at analysis and design of *“steel solutions to retrofit or upgrade vertical systems of existing reinforced concrete building and masonry buildings, in terms of ductility by the application of dissipative steel systems and in particular by eccentric steel bracings, steel shear panels/walls and BRB (buckling restrained brace) systems.”*

Within WP 3, VTT had the role of analysing ”*possible solutions using light gauge steel shear walls*” for the R.C. frame structures.

This study is continuation of previous studies on seismic retrofitting of reinforced concrete buildings by LGS elements. Numerical analyses (finite element method) are performed using a benchmark building modelled in SeismoStruct. The original geometry and loading of the benchmark building is presented in an earlier report (Fülöp, 2010). Focus of this report is retrofitting by light gauge steel plate shear walls and/or steel jacketing. First, SeismoStruct modelling is validated by comparing analysis models to available literature studies. Second, performance of several retrofitting techniques is tested on the benchmark building. Last, principles of detailing of the proposed retrofit solutions are given and discussed.

Authors

Contents

1 Introduction

This work continues the research on steel shear wall systems for seismic retrofitting of reinforced concrete frames done in STEELRETRO project earlier (Fülöp, 2010). The goal is to study the selected candidates for seismic retrofit more in detail. The focus is on the steel plate shear walls and column retrofit by steel jackets. In addition, principles for detailing the recommended solutions are provided increasing the usability of the results. Chapter 2 presents the comparisons of SeismoStruct modelling to the available literature studies. Chapter 3 shows the performance of the selected retrofit solutions on the benchmark building and Chapter 4 discusses the detailing of these solutions.

2 Verification of SeismoStruct modelling

SeismoStruct is a finite element (FE) analysis software capable of modelling, for example, reinforced concrete sections and steel profiles by dividing each finite element cross-section to many small fibres. The fibre model can only represent axial strength and shear effects are ignored.

2.1 External strengthening of columns

Existing concrete columns can be strengthened, for example, with steel jacketing or fibre wrapping. Test results on concrete columns strengthened with steel jacketing are available. First, a SeismoStruct model is created to reproduce the test results by Li et al. (2009). The objective is to achieve reasonable agreement with the test results, thus increasing the reliability of the SeismoStruct modelling of externally reinforced columns.

Li et al. (2009) tested reinforced concrete columns strengthened with steel jacketing and fibre-reinforced polymer individually, and also combining these two strengthening methods. They also published test cases without strengthening for reference. Figure 1 presents the column configurations tested by Li et al. (2009).

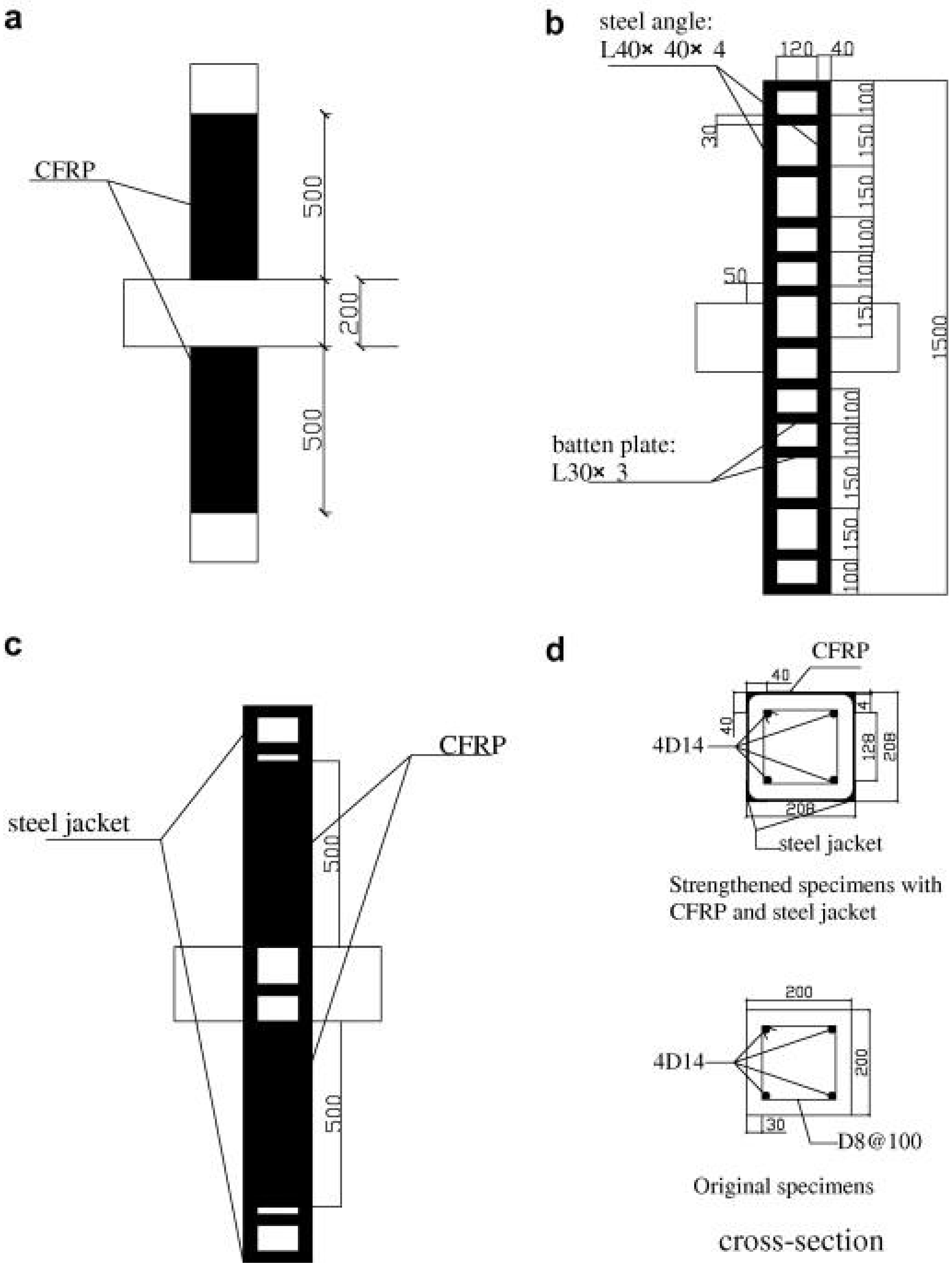

Figure 1. The configurations tested by Li et al. (2009).

Basically, four different configurations were tested:

1. Unstrengthened reinforced concrete (R.C.) column
2. R.C. column strengthened with steel jacketing
3. R.C. column strengthened with carbon fibre reinforced polymer (CFRP)
4. R.C. column strengthened with CFRP and steel jacketing

The differences between solutions 2 and 4 are not large, because the CFRP improves the confinement of the concrete, but no additional axial reinforcement is added.

The test specimens were subjected to a displacement spectrum presented in Figure 2.

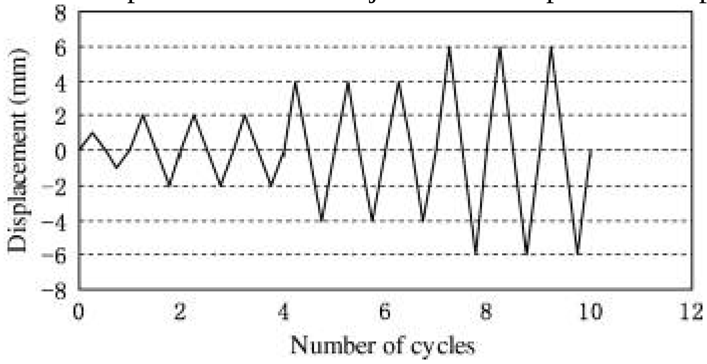

Figure 2. Displacement spectrum for the test specimens (Li et al., 2009).

Montuori & Piluso (2009) studied reinforced concrete columns strengthened with angles and battens subjected to eccentric load. However, the loading setup is more complicated to reproduce reliably with SeismoStruct, and thus these results were not reproduced.

2.1.1 Material and geometry modelling

The test specimen without any external strengthening (e.g. jacketing) is modelled in SeismoStruct using measured unconfined concrete compression strength, 44.8 MPa, for the cover and the confined concrete strength is increased to 57.7 MPa by method according to Paulay & Priestley (1992). According to this method effective confining stress f_l' for rectangular sections is given by

$$f_{lx}' = K_e \rho_x f_{yh},\qquad f_{ly}' = K_e \rho_y f_{yh}, \tag{1}$$

in the x and y directions, respectively. Here, f_{yh} refers to the yield strength of the transverse reinforcement bars and K_e is the confinement effectiveness coefficient, with a typical value of 0.75 for rectangular sections. The effective section area ratios of transverse reinforcement to core concrete, ρ_x and ρ_y, can be calculated using the area (A_b), spacing (s) and number (n_b) of the transverse rebars in both directions:

$$\rho_x = \frac{n_{bx} A_b}{s h_x},\qquad \rho_y = \frac{n_{by} A_b}{s h_y}. \tag{2}$$

The strength ratio between the confined core concrete and unconfined concrete column for symmetrically reinforced cross-sections is

$$K = \frac{f_{cc}'}{f_c'} = \left(-1.254 + 2.254\sqrt{1 + \frac{7.94 f_l'}{f_c'}} - \frac{2 f_l'}{f_c'}\right). \tag{3}$$

The strain at the peak stress of the confined concrete is

$$\epsilon_{cc} = 0.002\left(1 + 5\left(\frac{f_{cc}'}{f_c'} - 1\right)\right). \tag{4}$$

The above equation yields $\epsilon_{cc} = 0.0045$ for the test specimen without jacketing, which was used in the modelling. The nonlinear constant confinement model is used in SeismoStruct for modelling the concrete behaviour. The steel rebars and jackets are modelled using the bilinear model with kinematic strain hardening. Summary of the steel material properties used in the modelling is presented in Table 1.

Part	**Yield stress**	**Strain hardening parameter**	**Modulus of elasticity**
Longitudinal bar	384.77 MPa	0.005	200 GPa
Stirrups	326.95 MPa	0.005	200 GPa
Steel jacket and battens	338.5 MPa	0.005	200 GPa

Table 1. The material properties of steel parts.

The steel jacketing increases the existing concrete confinement and strengthens the column by introducing more load carrying members (L-profiles). Equations 1 – 4 can be extended to calculate the confinement effect of the L-profiles and battens (Montuori & Piluso, 2009). The confinement effect of the steel jacketing can be taken into account by adding the combining the confinement by battens and transverse rebars:

$$\rho_x = \frac{n_{bx} A_b}{s \boldsymbol{h}_x} + \frac{\boldsymbol{2} A_B}{s_B \boldsymbol{h}_{Bx}}. \tag{5}$$

Here, subscript “B” refers to the corresponding batten plate properties. Using equation (5) instead of (2) to calculate the confined concrete properties gives 73 MPa confined concrete compression strength for the test specimen.

In addition, Li et al. (2009) proposed a method for calculating combined confinement of steel jacketing and carbon fibre reinforced polymer (CFRP). They also applied the method for test specimens without CFRPs, which yields very similar formulations as equations 1 – 5. However, their formulations only take 35 % of the batten yield strength into account, which is apparently based on the test specimens, where failure was found on the batten-profile-connection. The failure of the batten plates is not the desired failure mode, because when utilized, the full longitudinal strength and ductility of the L-profiles can provide significant performance improvement on the column.

The column was assumed to be pinned at the top and the bottom and loaded as presented in Figure 3. The permanent load (vertical) on the top depends on the test specimen, while the horizontal load is introduced via time-history analysis with displacement control according to the spectrum in Figure 2. However, the repetitions of the same cycle were removed. The column is modelled using 10 elements, more densely concentrated around the middle of the column, where the plastic hinges will form. The external steel jackets are modelled using SeismoStruct repair elements, which are activated after the vertical compression force is applied. The steel jackets are represented using longitudinal reinforcement bars of equivalent area to the L-profile in a reinforced concrete section with very weak core concrete. Each element is modelled using 200 “fibres”.

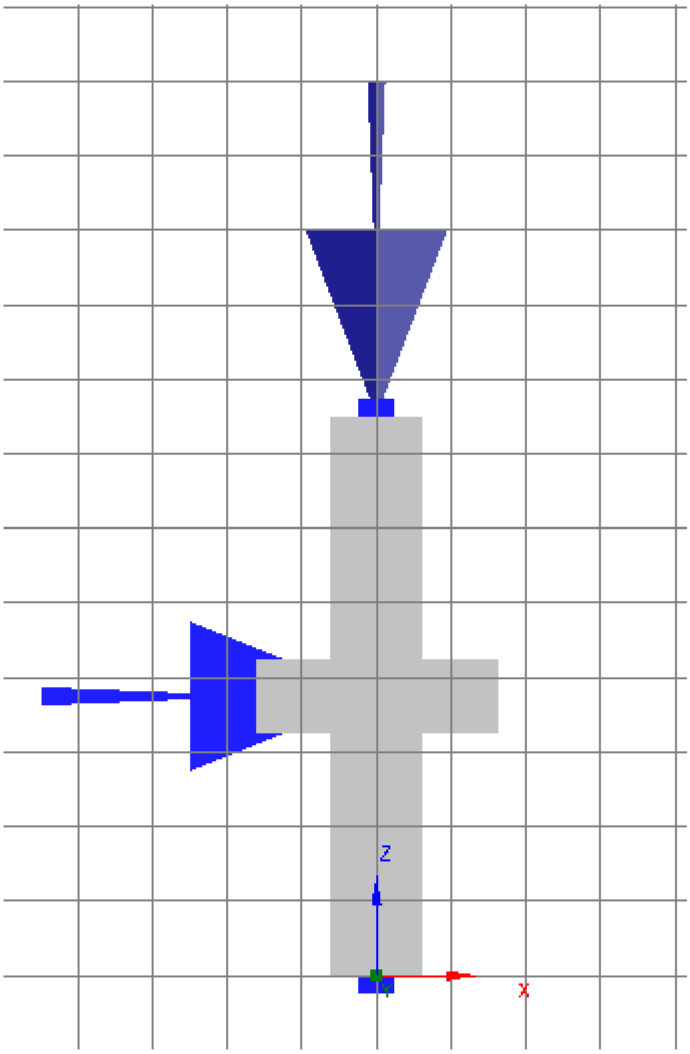

Figure 3. SeismoStruct column model.

2.1.2 Comparison to the test results

The test results for specimens

- A0, unstrengthened specimen without corrosion effects,
- B22, C22, corroded specimens with steel jackets,
- B221 and C221, corroded specimens with steel jackets and CFRP,

are reproduced in Table 2. Behaviour of specimens B22 and C22 is not possible to reproduce with SeismoStruct, because the batten plate connection to the L-profiles failed during the test and the full capacity of the L-profiles in the longitudinal direction was not utilized. The CFRP wrapping can be modelled in SeismoStruct only by taking into account the confining effect of the CFRP by increasing the confined concrete property.

Table 2 presents SeismoStruct analysis results for the maximum force. The corrosion effects present in the specimens are neglected for both analyses and the extra confining effect of the steel jacketing and the CFRP is not taken into account. However, reasonable agreement is found with the maximum force, which in this case is not largely affected by the concrete compression strength. The displacements are larger in the test specimens, which could be explained by the absence of clear yielding in the analysis model before the maximum force is reached. The SeismoStruct model does not take the rebar corrosion into account, which can cause deterioration of the bond between the concrete and the rebars, which changes the behaviour of the test specimens.

Table 2. Summary of the relevant test results and the corresponding analysis results.

Specimen	P_y (kN)	Δ_y (mm)	P_{max} (kN)	δ_{max} (mm)
A0	160.66	2.1	190.87	6.1
B22	220.21	2.75	265.79	8.17
C22	230.58	3.6	279.62	10.33
B221	265.68	2.73	316.45	9.95
C221	272.14	3.0	320.64	12.28
A0 Analysis			~172	~2.7
B22 / B221 Analysis			~340	~2.3

Figure 4 presents the hysteretic curves produced for the unstrengthened specimen (A0) and the strengthened specimen (B22/B221).

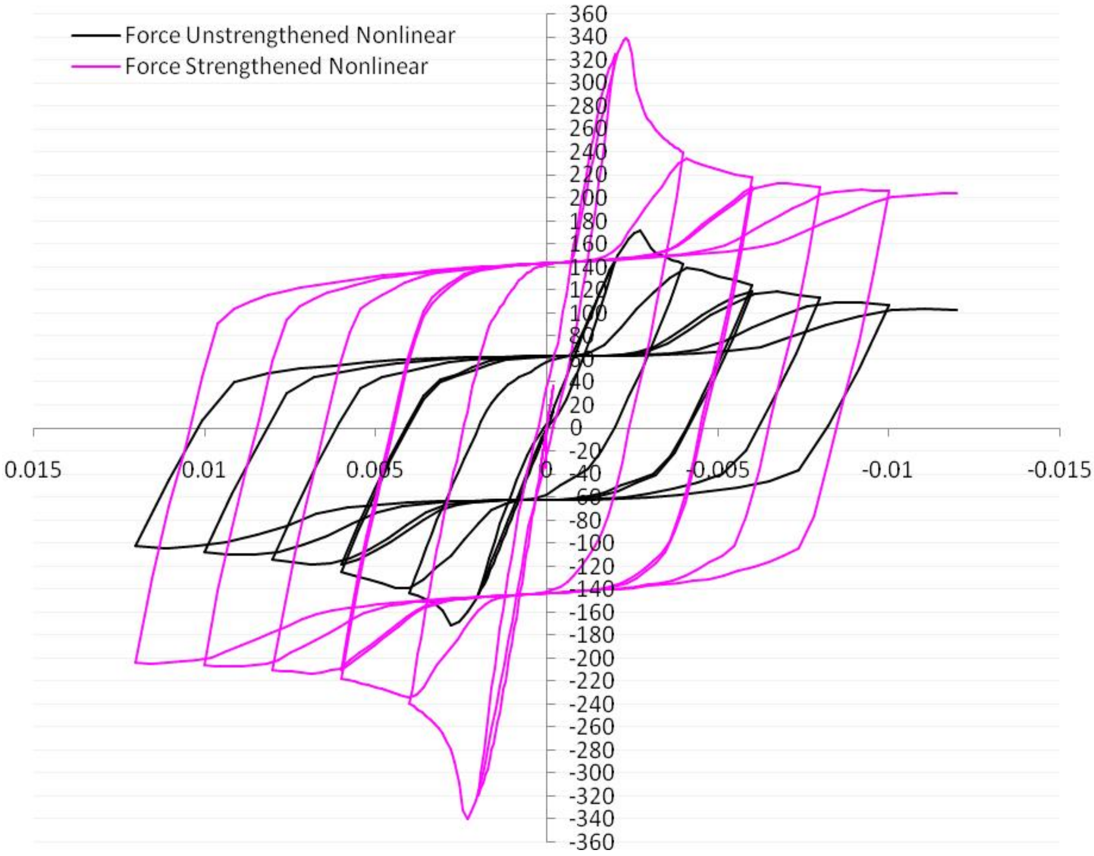

Figure 4. Hysteretic curves from the SeismoStruct analysis.

The confining effect of steel jackets was also tested by increasing the confined concrete strength from 58 MPa to 73 MPa, but this did not have significant effect on the response of the column in the test. This is because the core crush strength is only reached in one element even when the confining effect of the steel jackets is not taken into account.

In conclusion, reasonable agreement is found with the test results, which gives reliability to the SeismoStruct modelling, especially, in determining the maximum force the structure can withstand. The column model could be refined to take into account the corrosion effects, but the effect is assumed to be insignificant and therefore this was not done.

2.2 External steel frame with steel shear walls

Steel plate shear walls (SPSW) are extensively studied in the literature. The easiest method of modeling SPSW, without using complicated and costly shell element model, is to replace the steel plates with thin strips, i.e. “strip model”. The infill steel plate in the shear wall is relatively thin, which will cause buckling of the plate before yielding. However, Timler & Kulak (1983) proved that shear walls have a large post-buckling strength because of tension field effect. The tension field effect can be modelled using angled strips, which gives conservative results that can be used in design. This method was also preferred in the earlier report (Fülöp, 2010).

Driver et al. (1998) performed full scale experimental test on a four-storey steel frame with steel plate shear walls. The test setup is presented in Figure 5. This test is used in the literature to validate finite element models of SPSW. Shishkin et al. (2009) compared the original strip model (Timler & Kulak, 1983) and their own improved formulations to the experimental results by Driver et al. (1998).

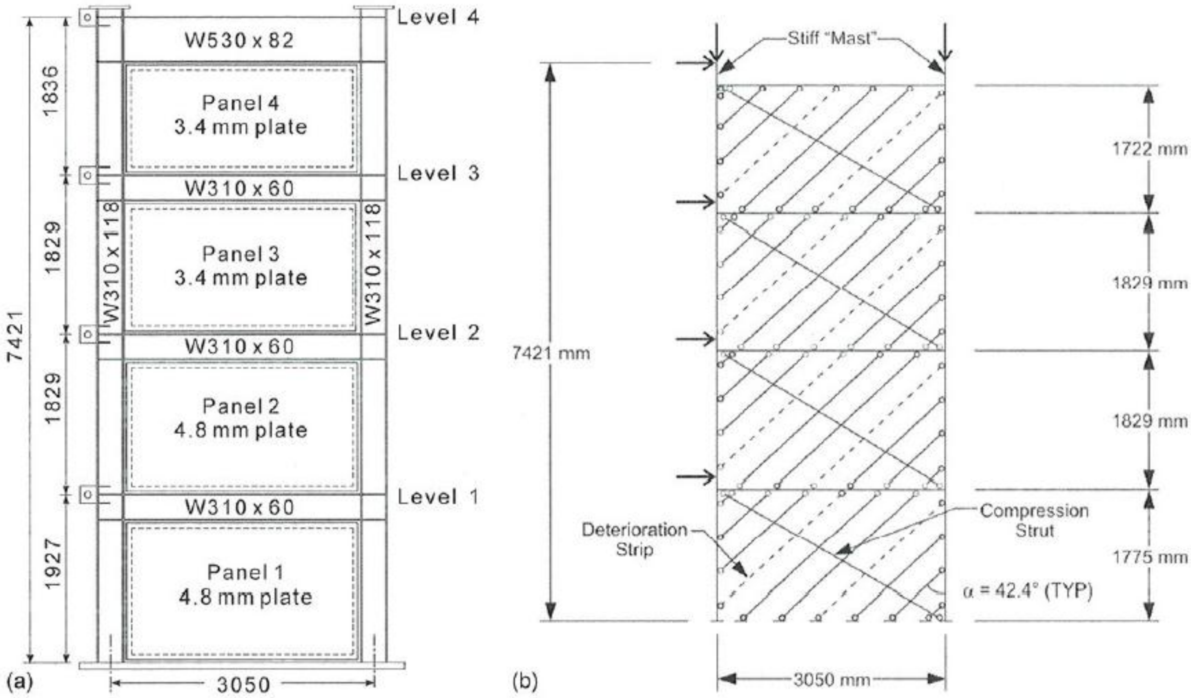

Figure 5. Four-storey steel plate shear wall experimental test setup and the strip model (Shishkin et al., 2009).

The compression and deterioration strips in Figure 5 are features of the advanced strip model (Shishkin et al., 2009) introduced to improve the performance of the model. The purpose of the compression strip is to improve elastic stiffness and ultimate strength of the model, which is underestimated with the basic strip model, because the basic strip model neglects the small increases in compression strength and stiffness caused by the infill plate. The deterioration strips model the weakening phenomenon rising principally due to the kinking of the infill plates in the corners, which can be modelled by removing the strips from the model when determined elongation is reached. Furthermore, plastic hinge nodes and so-called panel zone were inserted to the model to improve accuracy of the modelling. The panel zone refers to the beam-column connection, which is assumed to be very stiff, and therefore it is modelled with rigid elements in the advanced strip model. Figure 6 displays the test results for the 4-storey frame compared to the detailed model (shell elements), the advanced strip model and the basic strip model.

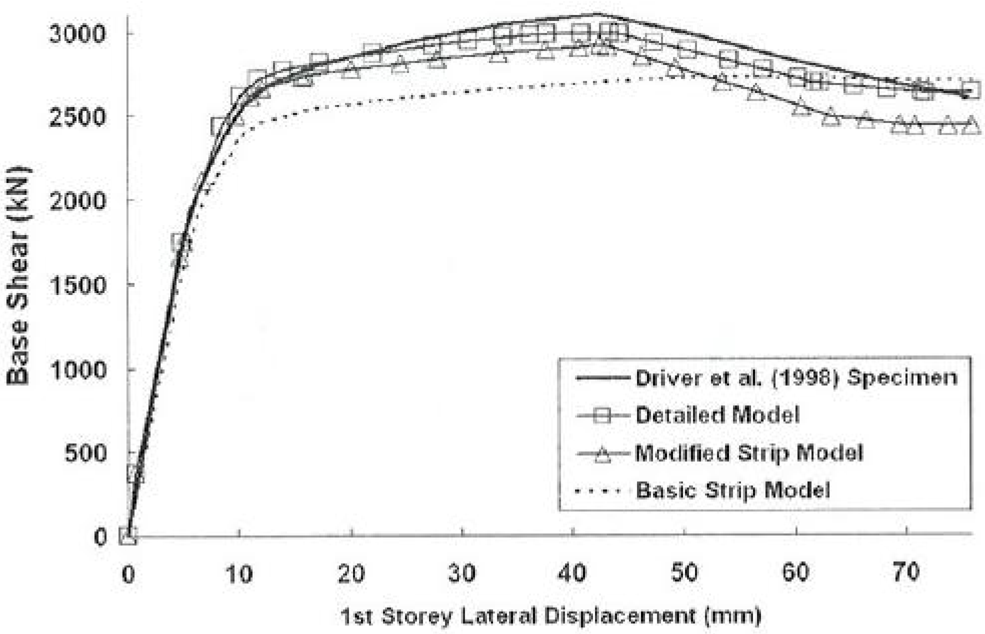

Figure 6. Response curves for test specimen, detailed model, advanced strip model and basic strip model (Shishkin et al., 2009).

This four-storey frame was also reproduced using the basic strip model and some features of the advanced strip model in SeismoStruct. The geometry of the frame was replicated using the information available in the article by Shishkin et al. (2009), because the original article by Driver et al. (1998) was not available. However, some simplifications and guesswork had to be done, because all the information was not available:

- the vertical load on columns was not known, several were tried,
- the horizontal pushover loads were applied to the nodes at the center of the beams,
- yield stresses of the materials were not known,
- the distance between the strips is not known,
- one infill plate is represented with one type of strip (corner strips “too large”),
- the lowest infill plate is pinned at the base,
- the frame is assumed to be fixed at the base.

The yield stress was assumed to be 275 MPa for the frame members and 235 MPa for the infill plates. Each infill plate was represented with 9 or 10 strips, as shown in Figure 7.

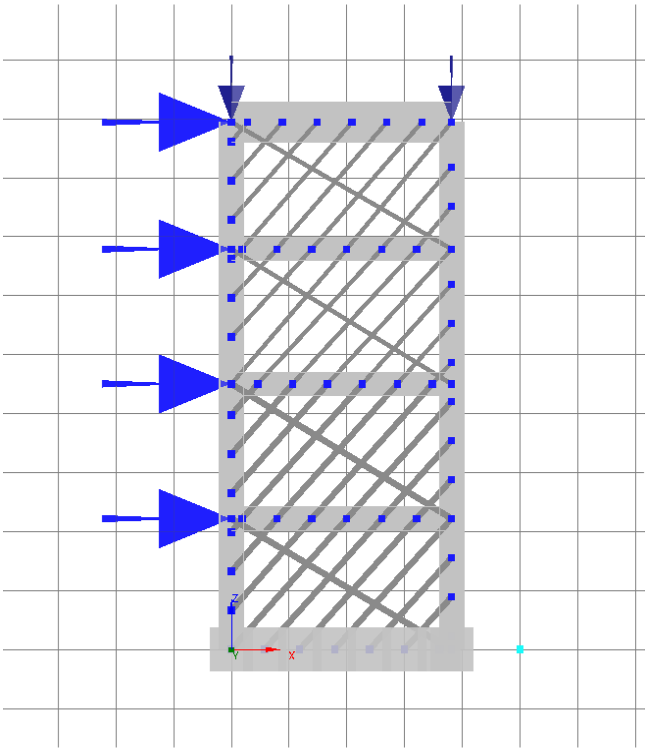

Figure 7. The test specimen modeled with SeismoStruct, including the compression strut.

A pushover analysis was performed using SeismoStruct varying the vertical load. The analysis used response control, where the first floor displacement was monitored until it reached 100 mm, or the analysis stopped because of convergence problems. Figure 8 presents the basic strip model pushover curves with different vertical loads and base fixing. The vertical load does not have major effect on the results, while fixed base provides large improvement. The basic strip model with fixed base provides similar results as in the work by Shishkin et al. (2009).

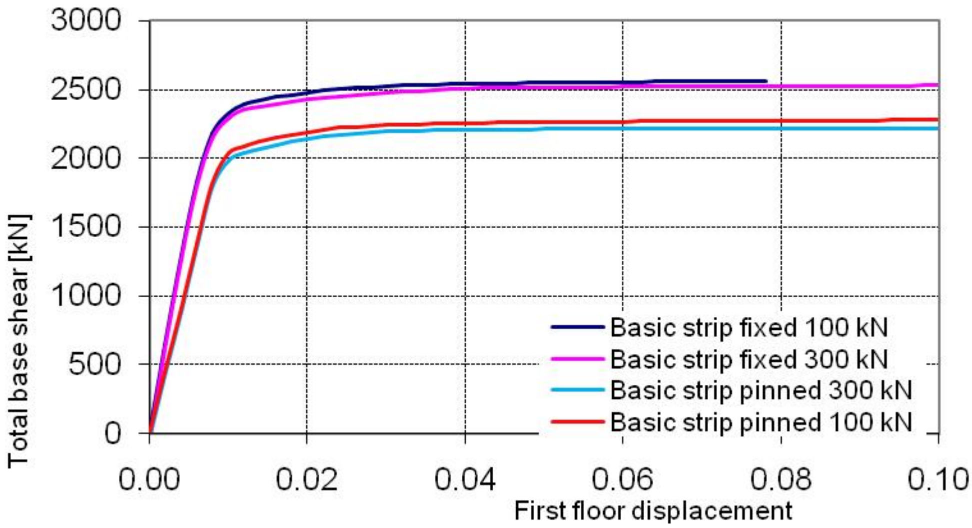

Figure 8. Pushover curves for the basic strip model with different vertical loads and base fixing.

Figure 9 displays the effect of the compression strut, which increases the stiffness and ultimate strength of the SPSW. Again, the ultimate strength of the SPSW is similar to the referred results. However, as the deterioration strips cannot be simply implemented in SeismoStruct and bilinear material model is used, the model is not able to catch the descending part of the pushover curve. The effect of vertical force on the maximum force remains small, however, convergence problems arise and probably ductility of the specimen is decreased.

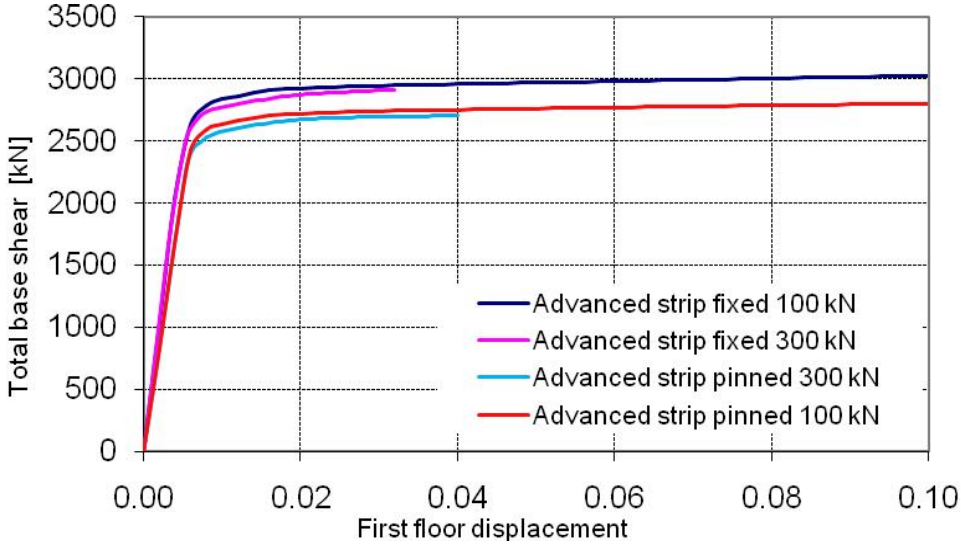

Figure 9. Pushover curves for the advanced strip model with different vertical loads and base fixing.

The results are in good agreement with the test results and other modelling techniques, considering the simplifications applied. Consequently, the basic strip model is used for the benchmar building, because it produces conservative results that are on the safe side.

3 Retrofitting technologies for the benchmark building

This report investigates two retrofitting techniques in detail for the case building, defined in the previous report (Fülöp, 2010):

- steel plate shear walls attached to the existing columns that are also retrofitted with steel jackets,
- steel plate shear walls on a external frame that is connected to the existing columns.

The geometry and loads of the building are assumed to be same as in the previous report, if not otherwise stated in this report.

3.1 Material modelling

Initially, the material model for the case building is adopted from the previous work (Fülöp, 2010). However, a few changes are made to the material models:

- the confining effect of the steel jacketing is taken into account, where relevant, by the method presented in equations 1 – 5,
- nonlinear material model is tested for the cover concrete,
- elements that are only subject to tension and compression are changed to truss type,
- force based element modelling is tested.

The nonlinear concrete model is performing more stably than the trilinear material model in SeismoStruct (version 5.0.3) when analysing single column or the whole building. The trilinear concrete model had convergence problems, which made it difficult to complete the pushover analysis. This lead to the conclusion that nonlinear model is recommended. The differences in the results are minor, as demonstrated later.

The elements acting in tension in the strip model for steel plate shear walls are switched to truss type, which improved the convergence of the model, while the results are practically identical with previous version.

3.1.1 Different modelling techniques

SeismoStruct provides force-based (FB) and displacement-based (DB) finite element formulations. With displacement-based formulation used initially, each structural member needs several elements for proper modelling of possible plastic hinges in the members, while the force-based formulation allows plastic hinges to be handled with only one element, where plastic hinge location is defined in pre-processing. The required computing time for the pushover analysis can be reduced by reducing amount of elements in the building, which is possible when several of the DB elements are replaced with one FB element. The pushover analysis results are reasonably similar with these two formulations, as can be seen from Figure 10.

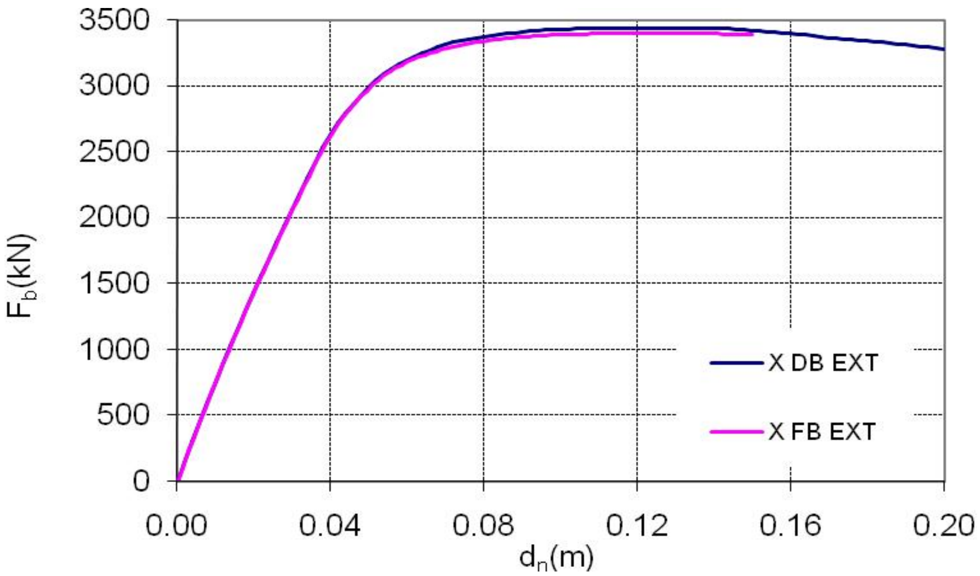

Figure 10. Displacement-based and the force-based FE formulations compared (EXT = external SPSW).

Unfortunately, we encountered several problems while using force-based element formulations in SeismoStruct version 5.0.3 (Build 2). SeismoStruct produced several types of runtime errors, which made it difficult to run the analysis successfully to the end. Two consecutive runs with the same settings could produce different errors, which is not an acceptable situation.

3.2 Steel plate shear walls between the existing columns

The previous work concluded that steel plate shear walls can be modelled using so called strip-model, where the post buckling tension field effect of the steel plates is modelled using strips of steel that respond only to tension.

Numerous advantages and disadvantages were found when the model of the building was retrofitted with strips corresponding to the steel plate shear walls. Clearly, the performance of the building improved, although desired seismic performance was not achieved in the x-direction. However, the structure responds to the earthquake loading by soft-storey mechanism, where the bottom columns fail and the upper structure remains comparatively untouched. This kind of catastrophic failure needs to be avoided. The columns should be strengthened to prevent the soft-storey mechanism. It is possible to improve performance by retrofitting columns attached to the shear walls with external steel jackets, which increase concrete confinement and provide bending resistance to the columns.

The steel plate shear walls increase the stiffness and the strength of the structure and also change the load distribution on the structure, which can cause problems. The beams and the columns connected to the shear walls must withstand large forces and moments. Especially difficult is the large uplift introduced in the column bases, which is very difficult to handle in foundation detailing. In addition, it is not evident how to fasten the steel infill plates to the relatively weak columns.

3.2.1 The effect of steel jacketing

The steel jacketing can stiffen the columns, increase concrete strength and provide enhanced ductility. The steel plate shear walls increase the loads on the columns next to them significantly; therefore, these columns are good candidates to be reinforced with L-profiles. Approximate upper limit for amount external steel jacket reinforcement can be determined by checking that reinforcement ratio

$$\rho_t = \frac{A_{st}}{A_g} \leq 0.04 \qquad (6)$$

is below the limit (Paulay & Priestley, 1992). Here, A_{st} refers to the total tension reinforcement area and A_g is the gross column area.

The initial analysis is performed using L100x5 L-profiles, which is the size used in the previous study for studying the effect of bracketing the 1st floor columns. In the first case, all columns connected to the steel plate shear walls are reinforced with the same L-profiles (L100x5).

The increase in strength on confined concrete inside the steel jacketing can be calculated using equations 1 – 5. This gives confined concrete compression strength approximately 33 MPa with 0.008 strain at peak stress. These values are used for modelling confined concrete when jacketed with L100x5 profiles using 100x5 battens with 400 mm spacing.

3.2.2 Results of the pushover analysis

Figure 11 presents the pushover curves for the building in x- and y-directions. The trilinear model refers to the trilinear model in the cover concrete, while nonlinear model was adopted for cover concrete to enhance convergence characteristics. The nonlinear cover concrete has a higher strength after crushing, which explains the higher stiffness in the pushover curves after initial failures in the model. However, more significant are the gains in ductility of the model, when the analysis does not end prematurely without reaching the maximum resistance of the structure. The core concrete is modelled with the nonlinear material model as before.

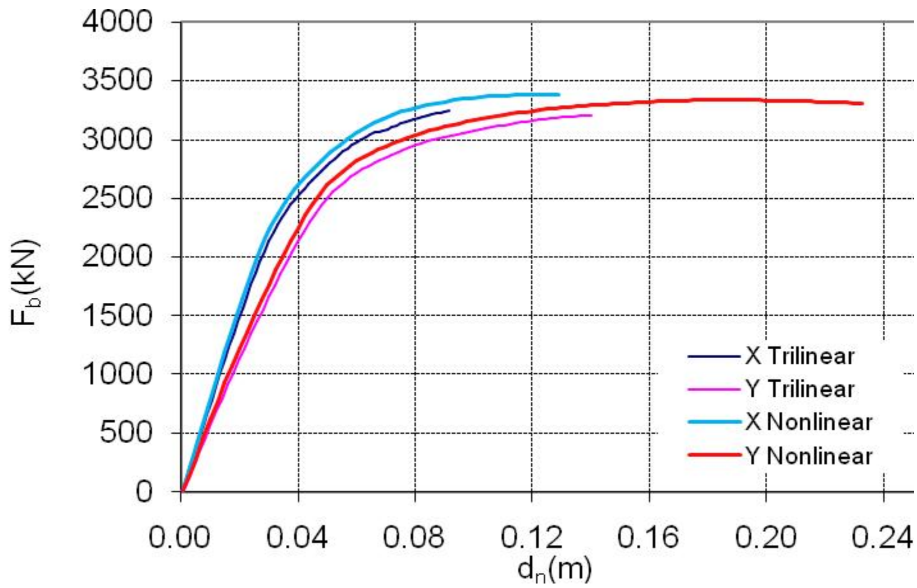

Figure 11. The pushover curves for the x- and y-directions.

The nonlinear model predicts much larger ductility for the building, mostly because using the trilinear cover concrete model presents convergence problems in the model. The ultimate strains for all steel parts are monitored and they are not reached before the maximum resistance of the building is reached.

Figure 12 illustrates the capacity and demand diagrams for x- and y-directions for the internal steel plate shear wall solution with nonlinear cover concrete material model. The building achieves desired performance level in both directions.

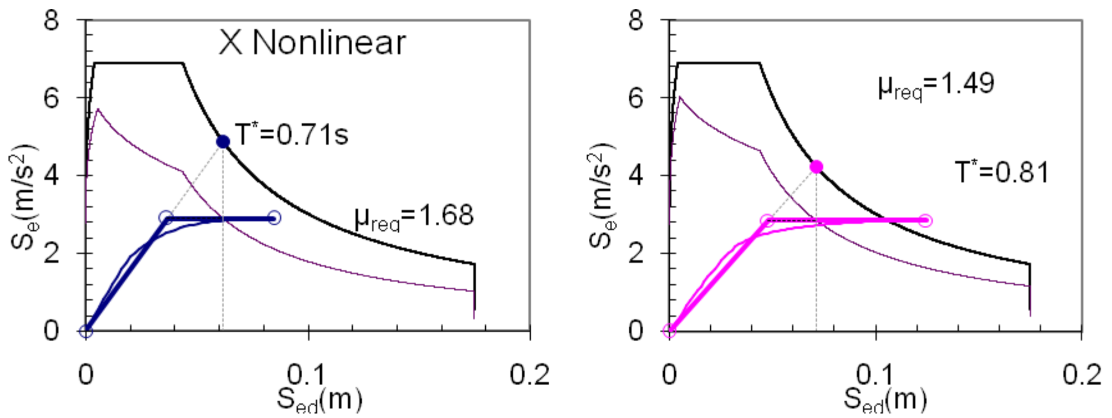

Figure 12. Capacity and demand diagrams for the internal shear wall system in x- and y-directions.

The steel jackets must be continuous through the floor slabs, when the full tension and compression resistance of L-profiles needs to be utilized, as modelled, which can introduce difficulties in the execution in the existing structure.

Large compression forces are present in the first floor columns generated by gravity loads, nevertheless, the SPSW solution causes tension on the R.C. columns connected to the shear wall, which causes problems on the foundation. Table 3 presents the axial forces on the selected column bases next to the shear walls at the moment of maximum base shear of the building.

Table 3. The axial forces on the selected column bases in x- and y-directions.

Pushover Direction	C211 (kN)	C311 (kN)	C112 (kN)	C113 (kN)
X	240	-1500	-280	-250
Y	-470	-360	1210	-2160

Large tension (positive) forces are present on C112 column in pushover analysis in the y-direction, while the compression in C113 column also increases significantly. The situation is reversed, when x-direction pushover is performed, but the forces are smaller because the shear wall is wider. In addition, the external steel jacketing is heavily tensioned (240 kN) during the analysis, because it is not stressed at the beginning of the analysis (retrofitted to the building).

Figure 13 shows the amplified deformed shape at the end of the pushover analysis in x- and y-direction.

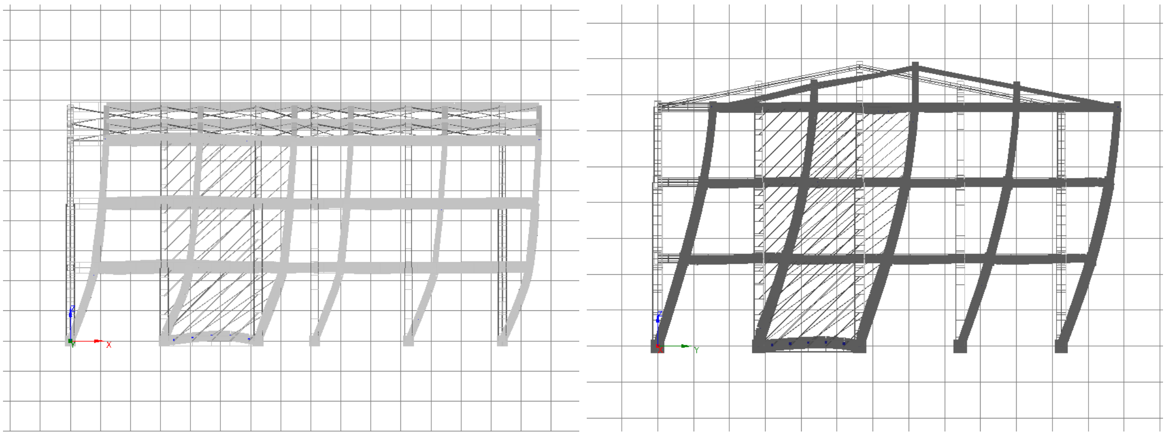

Figure 13. Amplified deformation pattern at the end of the analysis in the x- (left) and y- (right) directions.

The soft-storey mechanism is still present on the building in the x-direction, and further reinforcements to the other columns are needed to prevent this.

3.3 Steel plate shear walls on an external steel frame

Shear walls on external frames connected to the building are investigated to counter the problems found with the existing columns interacting with the shear walls. While some problems might be solved, new problems emerge, such as connections between the external frame and the existing concrete structure. The external frame with a shear wall should behave similarly as the shear wall solution between the original columns. However, the external frame, if large enough, might bring some extra stiffness and strength to the building.

The external frames are positioned at the same locations as the shear walls in the previous solution, except 0.5 meters outside from the middle point of the outermost columns. However, the distance between the building and the steel frame does not influence the analysis results, because the connections between the steel and concrete frame are modelled as nodal constraints (rigid link) active in x- and y-direction. The z-direction (vertical) is not constrained avoiding the vertical (gravity) load transfer from the building to the external frame. The external columns are constrained at each R.C. column beam intersection to the building.

3.3.1 Results of the pushover analysis

The initial columns and beams for the external frames were selected using engineering judgement. The most loaded members are the bottom columns and the beam at the ground floor. The selected columns are IPE500 and the beam is IPE600. The second and third floor columns are IPE400, while beams from second to third floor are IPE400 and IPE300. The shear wall properties are same as before. Figure 14 presents the pushover curves for the building strengthened with the external frames. Again, trilinear model refers to the trilinear cover concrete model, while core concrete is nonlinear in all models.

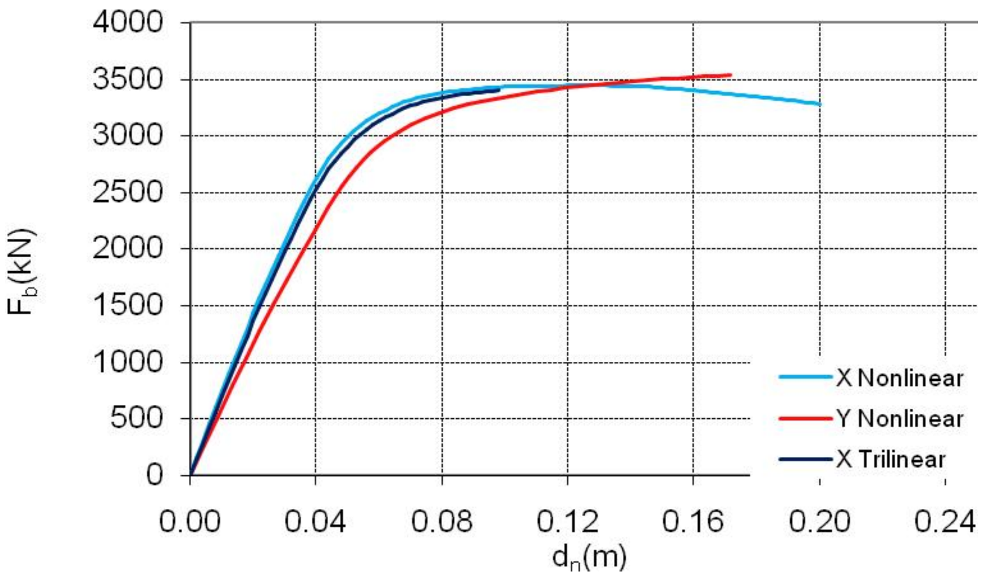

Figure 14. Pushover curves in x- and y-direction for the building with external shear walls.

The pushover in x-direction has numerical problems and the maximum force and displacement is not reached with trilinear cover concrete model. Although the trilinear model also meets the desired performance, the nonlinear cover concrete model is used for demand and capacity analysis, because fewer problems came across in the analysis. The demand and capacity diagrams to the x- and y-direction can be calculated from the pushover curves and the results are displayed in Figure 15.

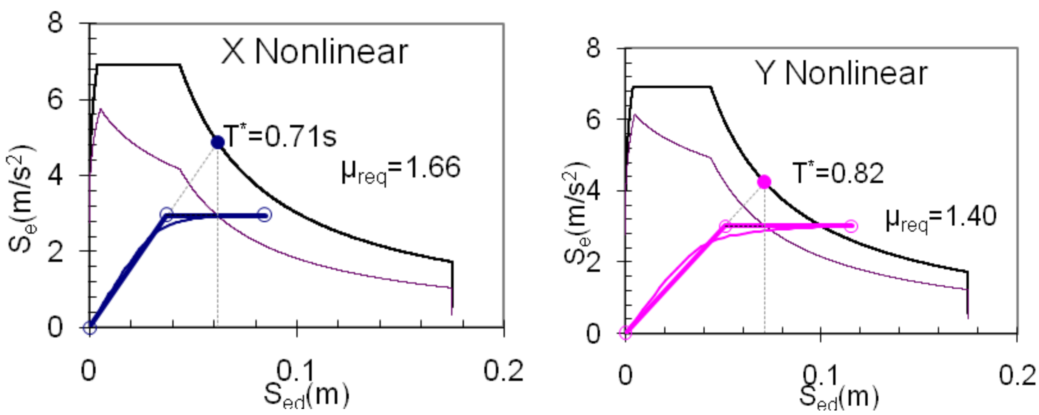

Figure 15. Capacity and demand diagrams for the external shear wall system in x- and y-direction.

Figure 15 shows that seismic requirement level for capacity of the building is reached in both directions and in y-direction there is quite much extra ductility.

During the analysis, the bottom columns in the external frame on the tension side were yielding quite early. The external frame, unlike the concrete building, does not have large compressive forces on the columns, which means that very large tension forces start to build up on the columns where the shear wall is pulling. Table 4 presents the reaction forces at the selected column bases in the external frame.

Table 4. Axial forces on the selected external column bases in x- and y-direction pushover analysis.

Pushover direction	Ce211 (kN)	Ce311 (kN)	Ce112 (kN)	Ce113 (kN)
X	1260	-1990	13	-12
Y	-30	50	2020	-2060

In the y-direction, the column next to the shear wall has a large uplift force (2020 kN), which will be very difficult for the foundation design. The external walls positioned resist x-direction actions have negligible amount of forces when y-direction is studied, but the situation is reversed when direction of loading is changed, which is to be expected because the shear walls are flexible in the minor direction.

By preventing external frame yielding the strength of the whole system can be increased, possibly leading to very big profiles. However, this does not solve the problems with large uplift forces on the foundations (actually worsening the situation) and the connections between the external frame and the building would become even more difficult.

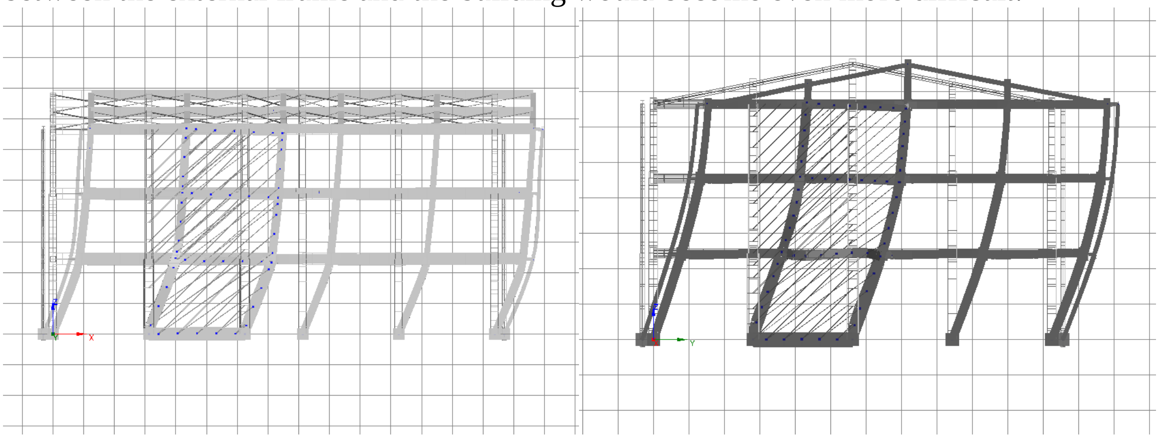

Figure 16. Amplified deformations at the end pushover analysis in x- (left) and y- (right) directions.

As expected, the results are similar with internal shear wall solution, and here also the x-direction is more problematic, because the soft-storey mechanism forms at the bottom columns.

It is possible to increase the size of the external frame, which might increase the performance of the building, which will also increase the foundation requirements. The columns are changed to IPE600 – IPE500 – IPE500 from down to bottom and beams to IPE600 – IPE500 – IPE400 and IPE400, also from down to bottom. This increases the strength of the frame, while ductility remains similar as can be seen from Figure 17.

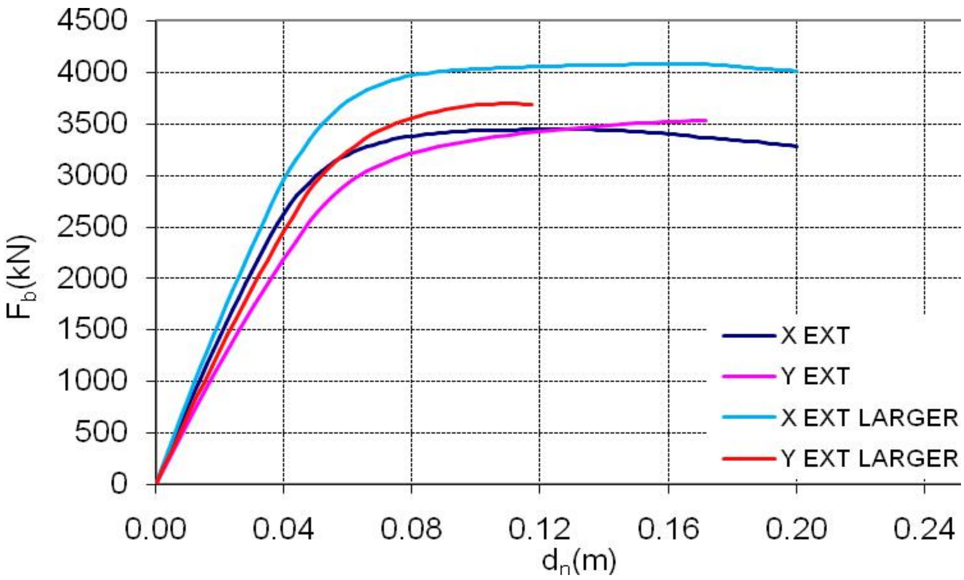

Figure 17. Pushover curves from two external frame analyses compared.

The increase in strength is naturally visible in the bases of the external column, where larger forces appear. This suggests that additional strength of the frame might not be utilized before the column bases fail, or the connection between the external frame and the building fails.

When detailing is considered, it is easier to select constant column cross-section, because it might be possible to make the column from a single member, thus decreasing the amount of connections connecting different members. In addition, it might not be economical to select many different profiles, although some gains in weight are achieved. Therefore, constant cross-section columns were also tried and only one beam size was used. The column is selected to be HE450A, which has similar height to the concrete column width, helping with the connections. The beam cross-section (IPE600) is also selected to match the 600 mm high concrete beams.

Figure 18 illustrates the gains with achieved with a large constant cross-section columns and large beams. The results are similar with the previous models, except with a quite large increase of capacity in the y-direction.

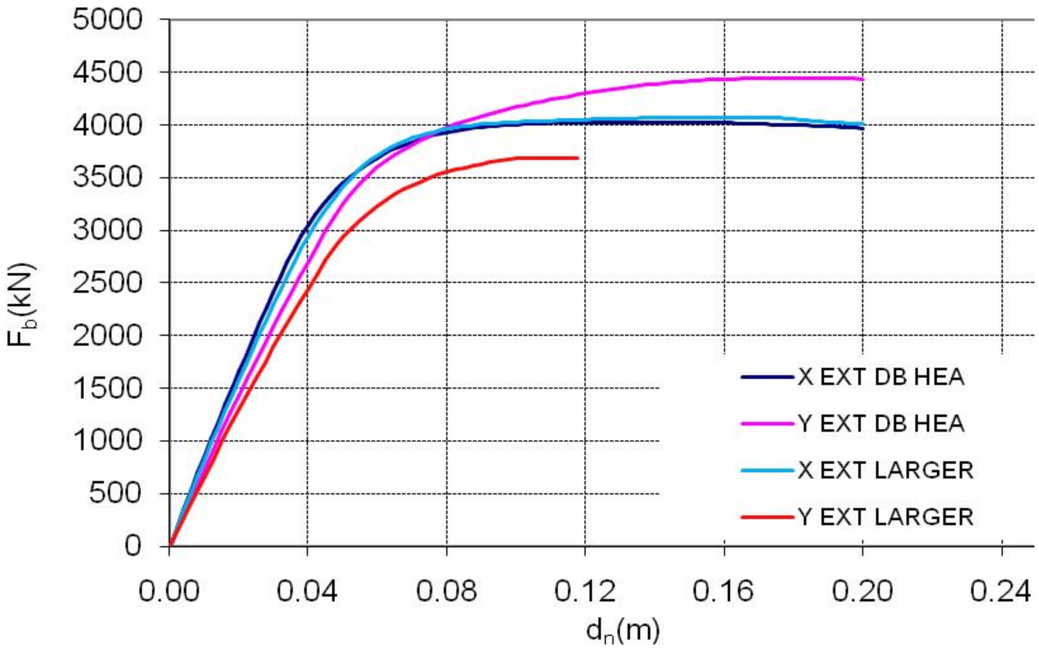

Figure 18. Larger version of the IPE-column solution compared to the constant cross-section HE450A columns.

A novel solution for the external shear wall is to replace the I-profiles with U-profiles, which helps with the connection design. The connection details are discussed in a later chapter. Figure 19 shows the pushover curve comparison between the U-profile and the strongest I-profile solution. Clearly, the I-profile solution offers better performance, but the ductility of the U-profile frame is also satisfactory. However, the SeismoStruct model does not take into account possible connection problems with either solution. Presumably, it is difficult to connect the I-profile frame to the existing concrete as stiffly as the model predicts which leads to the analysis overestimating the stiffness and strength.

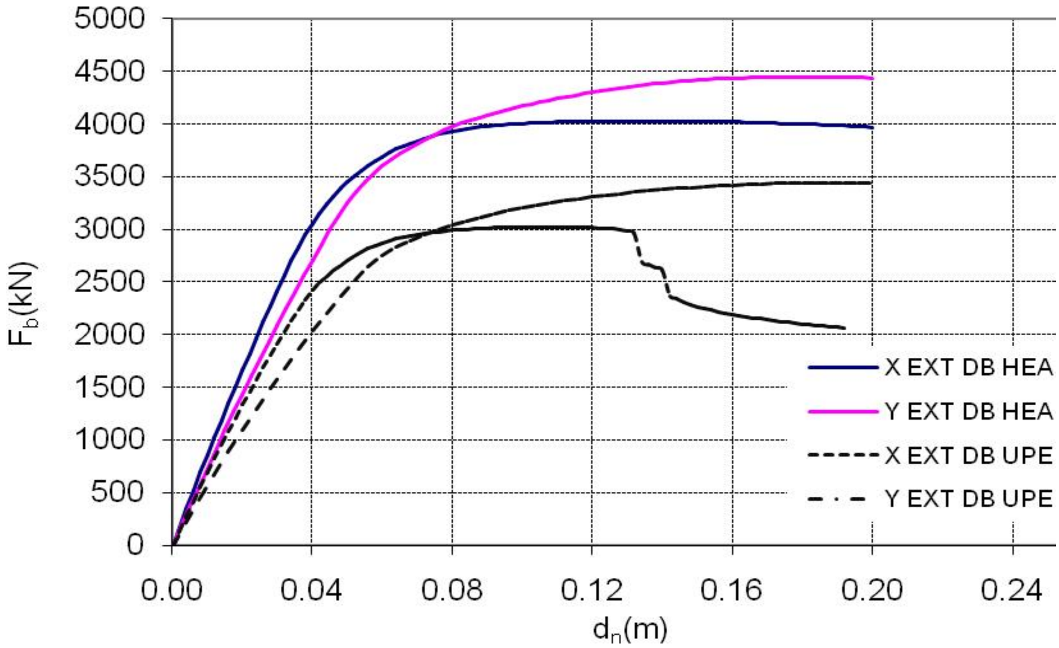

Figure 19. Pushover curves for external frames with U-profiles and I-profiles.

3.3.2 Comparison of the shear wall solutions

Figure 20 illustrates the differences between external and internal steel plate shear wall configurations. These two typical configurations have the same shear wall, and therefore the differences in behaviour can be explained by the amount of steel added to the external frame. The external frame shear wall provides slightly more strength and similar ductility with the internal shear wall.

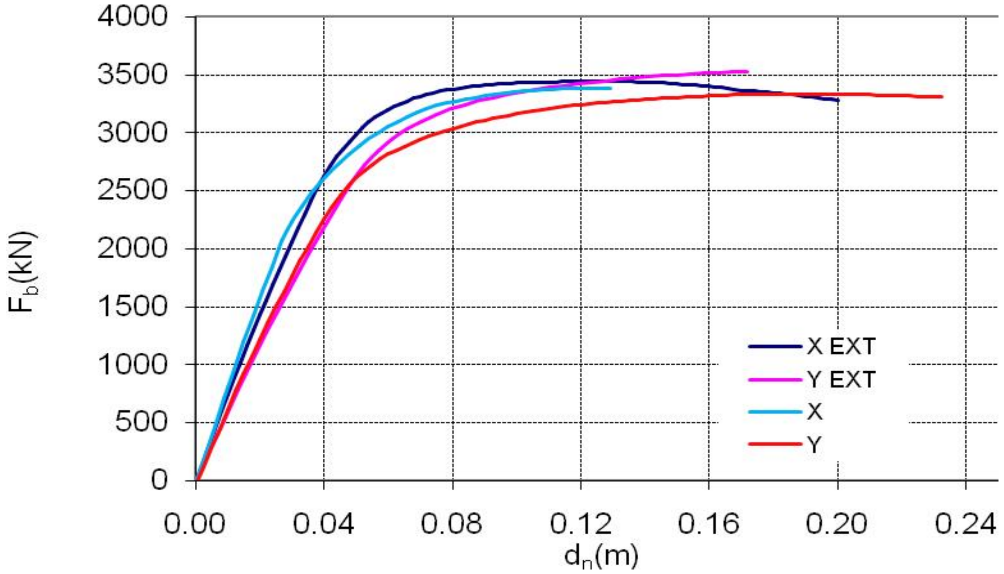

Figure 20 .External and internal shear wall pushover curves in x- and y-directions.

3.4 Perforated steel plates and aluminium steel plates

Steel plate shear wall can impose large loads on the surrounding members. In addition, thin steel plates might not be available everywhere. Therefore, regularly perforated steel plates have been researched as an option by Purba & Bruneau (2009). Furthermore, perforations allow utility systems to pass through the infill plates, if necessary. Figure 21 presents a possible layout and approximate size of the perforations. By changing the size and number of perforations the strength of the shear wall can be adjusted in a very flexible way.

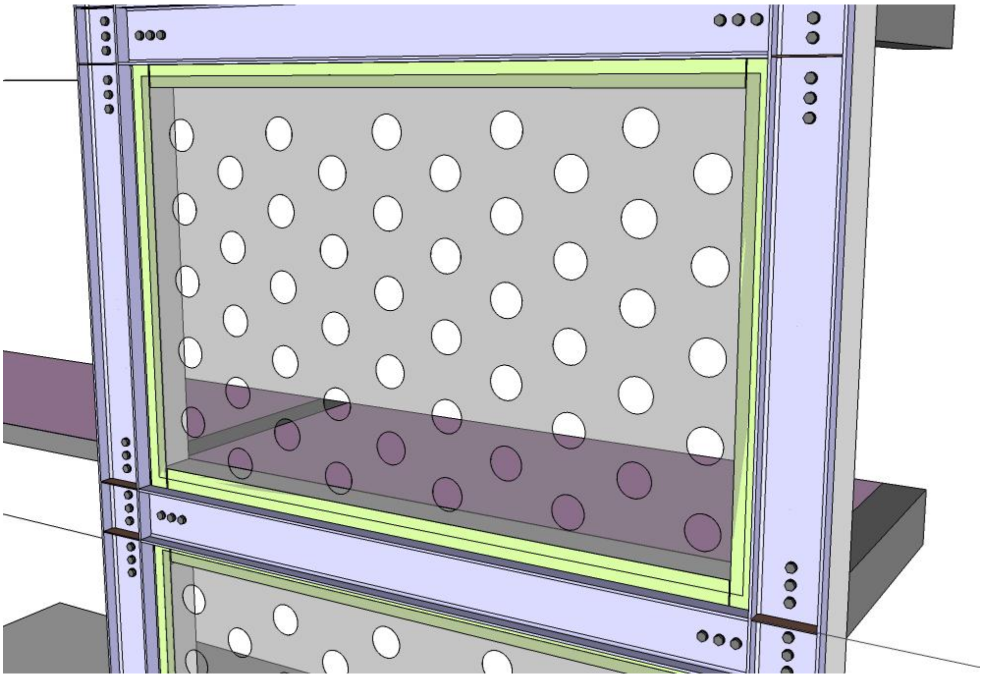

Figure 21. Perforated steel plate shear wall.

Purba & Bruneau (2009) proposed an equation for evaluating the strength of the perforated steel plate shear wall compared to a shear wall without holes:

$$V_{yp,perf} = \left[1 - 0.70\frac{D}{S_{diag}}\right]V_{yp}, \tag{7}$$

where D is the diameter of holes, S_{diag} distance between the centers of holes and V_{yp} strength of the shear wall without perforations. Equation (7) is valid for geometries having a perforation ratio D/S_{diag} between 0.12 and 0.71.

Using the maximum perforation ratio (0.71) allowed for equation (7) the SPSW can be analysed in SeismoStruct by reducing the LGS thickness by 50.3 %. The advantage of this is the reduced demand on boundary members of the SPSW and the foundations. The foundations are modelled fixed in y-direction and pinned in x-direction in other analyses in this report, however, spring foundations are applied with UPE frame having perforated LGS shear walls. The springs are calculated by CERI (University of Rome) and assume that micropiles are inserted to retrofit the foundations. The spring properties are presented in Table 5.

Table 5. Spring properties for foundations.

Spring type	K_x (kN/m)	K_y (kN/m)	K_z (kN/m)	$K_{\theta x}$ (kNm/rad)	$K_{\theta y}$ (kNm/rad)	$K_{\theta z}$ (kNm/rad)
T11	2740	9570	217330	49900	950	2400
T21	36350	11840	393460	89631	7920	27030
T13	34540	47730	478340	128830	61730	27100

T11 foundations are used in most of the R.C. columns excluding R.C. columns next to the shear walls. T21 springs are inserted in the R.C. columns next to the shear walls resisting x-direction pushover, while T13 springs are in the R.C. columns beside shear walls mostly resisting y-direction actions.

Figure 22 presents the demand and capacity curves for the perforated LGS shear wall in the UPE frame with spring foundations, showing that performance is satisfactory.

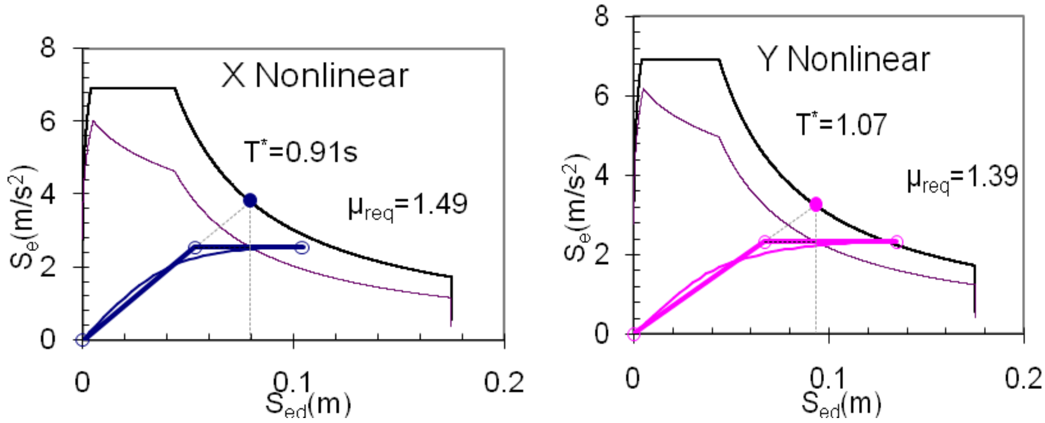

Figure 22. Demand and capacity curves for UPE frame with perforated LGS shear wall.

Above configuration also reduced the load on the foundations. The largest uplift in the foundations appears in y-direction pushover (926 kN), which is over 40 % reduction over the non-perforated UPE frame SPSW solution. By controlling the SPSW strength, excessive overdesign can be avoided while keeping the performance above the desired level.

Mazzolani (2008) tested and analysed different retrofitting methods on parts of a real building. In conclusion, the hysteretic properties of aluminium lead to good performance. The advantage of aluminium is the low yield with good energy absorption, while sufficient performance can be provided.

4 Principle detailing of the retrofit solutions

Successful manufacturing and on-site assembly of the retrofit solutions is not necessarily straightforward, which highlights the importance of detailing. Poor detailing might result in increased cost and inadequate performance. This chapter presents illustrative examples of principles that can be used to make detailed drawings.

4.1 Principle detailing of the steel jacketing and SPSW between R.C. columns

Steel infill plates between the existing jacketed concrete columns impose large forces to the batten plates and the L-profiles. When the infill plate is divided to strips, the force in one strip is around 250 kN after yielding. The force of one strip can be idealized to be taken by one batten plate. In addition, these forces must be sustained without extensively crushing the weak existing concrete. Figure 23 presents an isometric view of the proposed retrofit system between the existing columns.

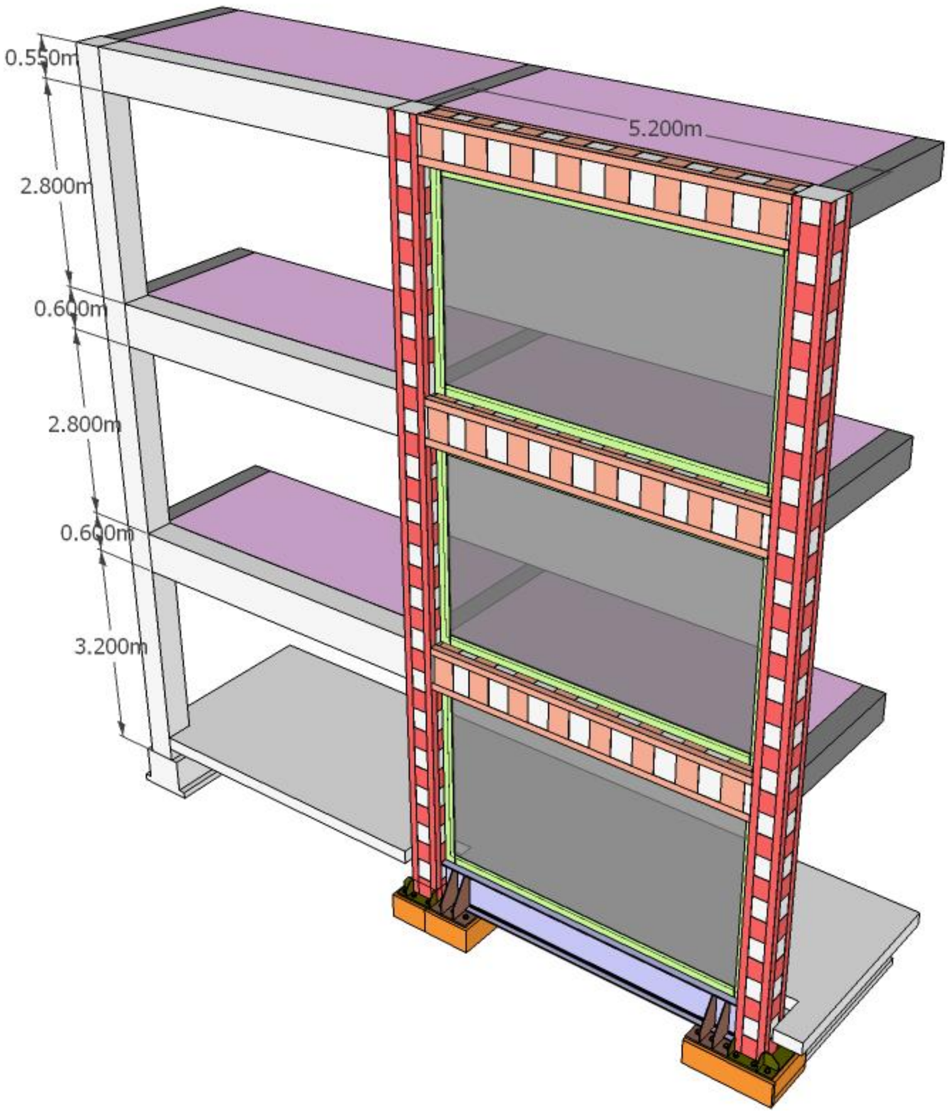

Figure 23. Isometric view of the proposed shear wall solution between the existing columns.

Figure 24 presents a basic batten plate and L-profile configuration, where the infill plate is welded to fin plate, which is then connected to the batten plates. In this case, the batten plate thickness and L-profiles would become uneconomical when the bending of the batten plate is considered coming from the strip force. Therefore, bending of the batten plates should be transformed to tension. Novel solutions must be tested to overcome these problems.

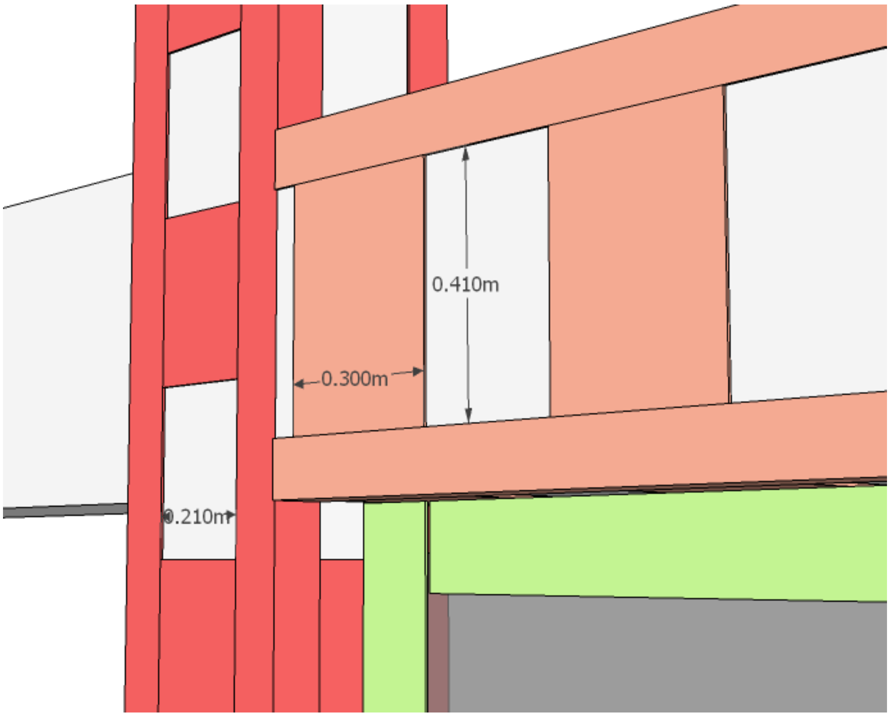

Figure 24. Infill plate connection detail, when it is between the existing columns.

Figure 24 illustrates also another difficulty related to the steel jacketing: the jacketing must be continuous through the concrete beams and floor slabs when the full benefit of the tensile properties of the steel profiles needs to be utilized. In addition, the steel jackets must be connected to the base plate.

Figure 25 illustrates two alternative connection details for connecting the infill plate to the concrete columns. The key idea in these solutions is to prevent the bending of the batten plate and instead take the forces from the infill plate mostly as axial tension/compression. However, it is very difficult to verify the performance of these solutions without testing.

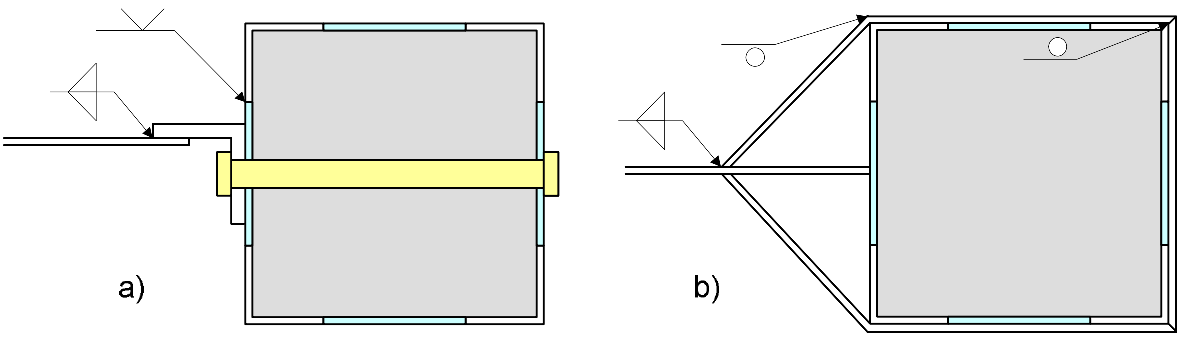

Figure 25. Alternative connection principle for the infill plate: a) anchored through the concrete column and b) tied with plate around the column.

The anchored solution, (a) in Figure 25, needs modifications to the existing concrete and the L-profile causes slight eccentricity. On the other hand, the second solution (b) is quite difficult to install and the thickness of the plates providing adequate performance might become too thick for feasible construction.

Another alternative solution exists that has been used with steel bracings on concrete frames. The idea of the solution is construct a rectangular steel frame between the concrete columns and beams. This kind of frame has been previously used for creating steel bracing systems for seismic retrofitting. Instead of bracings, a steel plate shear wall could be connected to the steel frame. The steel frame is typically connected to the concrete members by adhesive anchors. However, Ohmura et al. (2009) developed and tested a new connection method without anchors, which does not require drilling in the existing concrete, as is shown in Figure 26.

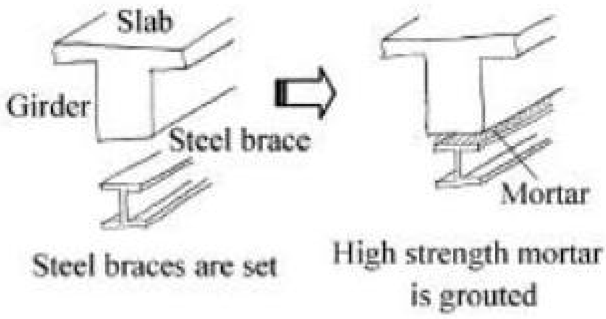

Figure 26. The new connection method without anchors developed by Ohmura et al. (2009).

Figure 27 illustrates the complex situation at the base of the concrete column. The uplift caused by the shear walls during earthquake poses difficulties in upgrading the foundations and tying the steel jacketing to the base plate. It is to be expected that the original concrete foundation cannot sustain large uplift forces without failing, which calls for large rework of the foundations around the original column. In addition, the steel jacketing needs to be tied to the base plate so that the tensile properties of the L-profiles can be utilized.

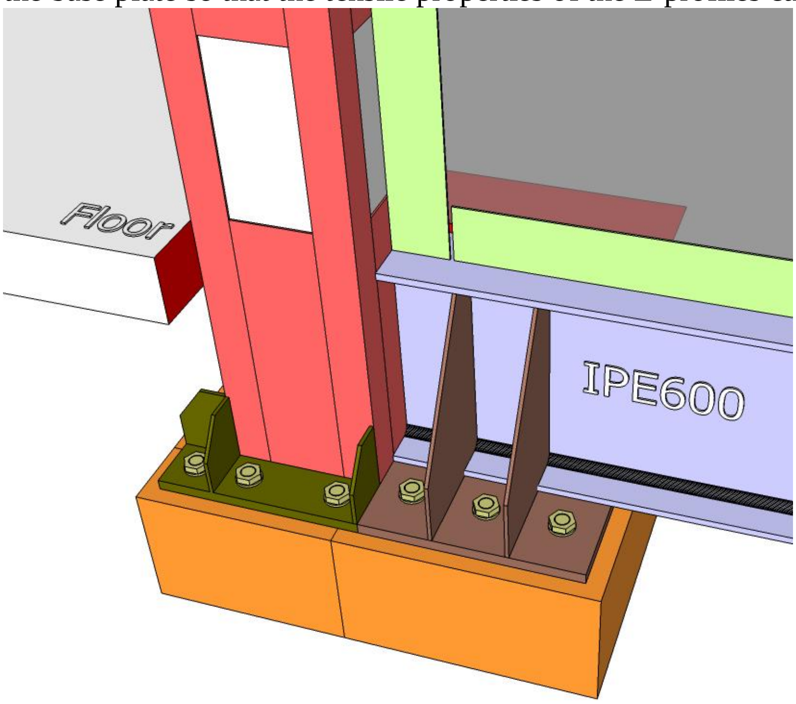

Figure 27. Proposed solution principle for enhancing concrete column base and base beam connection.

4.1.1 Installation

Many issues remain, even if the problems with connection details between the infill plate and the concrete columns are solved. Assembly and installation of the whole retrofit system is complicated. Large parts of the original concrete foundations, floors and walls must be demolished and replaced with new parts. The steel jacketing must be continuous through the floors and concrete beams, which requires that a lot of material must be removed. Furthermore, the concrete beams must be jacketed in order that the infill plates can be connected, which means that the walls must be demolished where the infill plates would be. Major on-site welding operations are needed, because most of the parts cannot be installed otherwise. In conclusion, major rework operations are needed and large steel parts must be installed around or through the columns providing a reasonably strong retrofit system, which probably proves to be costlier than the external shear wall solutions proposed in the next chapter.

4.2 Detailing of the external SPSW and connections to existing structure

The external shear wall solution relies on standard construction practise. The columns can be made of continuous profiles, if the limits from transportation are not reached. The beams can be welded to the columns, or bolted connections used if stiffness of the connection is verified. The usual solution for the infill plate connection includes fin plates welded to the beams and columns, which are then welded or bolted to the steel infill plate, as shown in Figure 28. Schumacher et al. (1997) studied different connection methods, also proving the performance of standard fin plate setup with testing. The strap plate is recommended for reinforcing the corner detail. The whole 4-storey shear wall solution can be manufactured at the machine shop, if the transportation is not an issue. If thin light gauge steel plate is used, welding can be problematic and bolted connection can be a viable alternative.

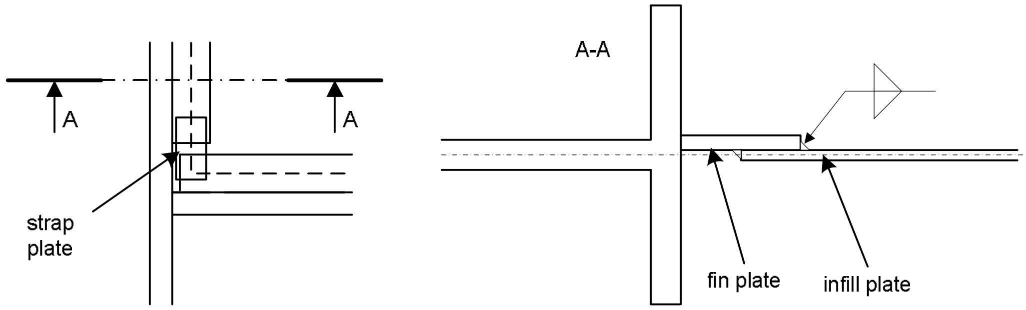

Figure 28. Conventional detail of the fin plate connection to infill plate with welding.

A stiff frame surrounding the infill plate ensures that the tension field develops fully, while a flexible frame decreases the capacity of the shear wall. However, the frame has to be connected to existing concrete that is relatively weak and large forces must be transmitted from the building to the shear walls. This introduces many problems that need to be solved in order that the retrofit solution can be effective.

Figure 29 illustrates the external shear wall placed next to a part of the existing structure. The external frame is manufactured of IPE600 beams and HE450A columns, which have dimensions close the concrete beams (400x600) and columns (400x400) helping with the connection design. Effectiveness of this solution depends largely on force transfer between the concrete and steel frames. As pointed out earlier, the existing concrete is not very strong, which means that careful detailing is needed. Ideally, the external frame is placed close to the concrete structure, because larger distance increases the required bending stiffness and strength in the connections.

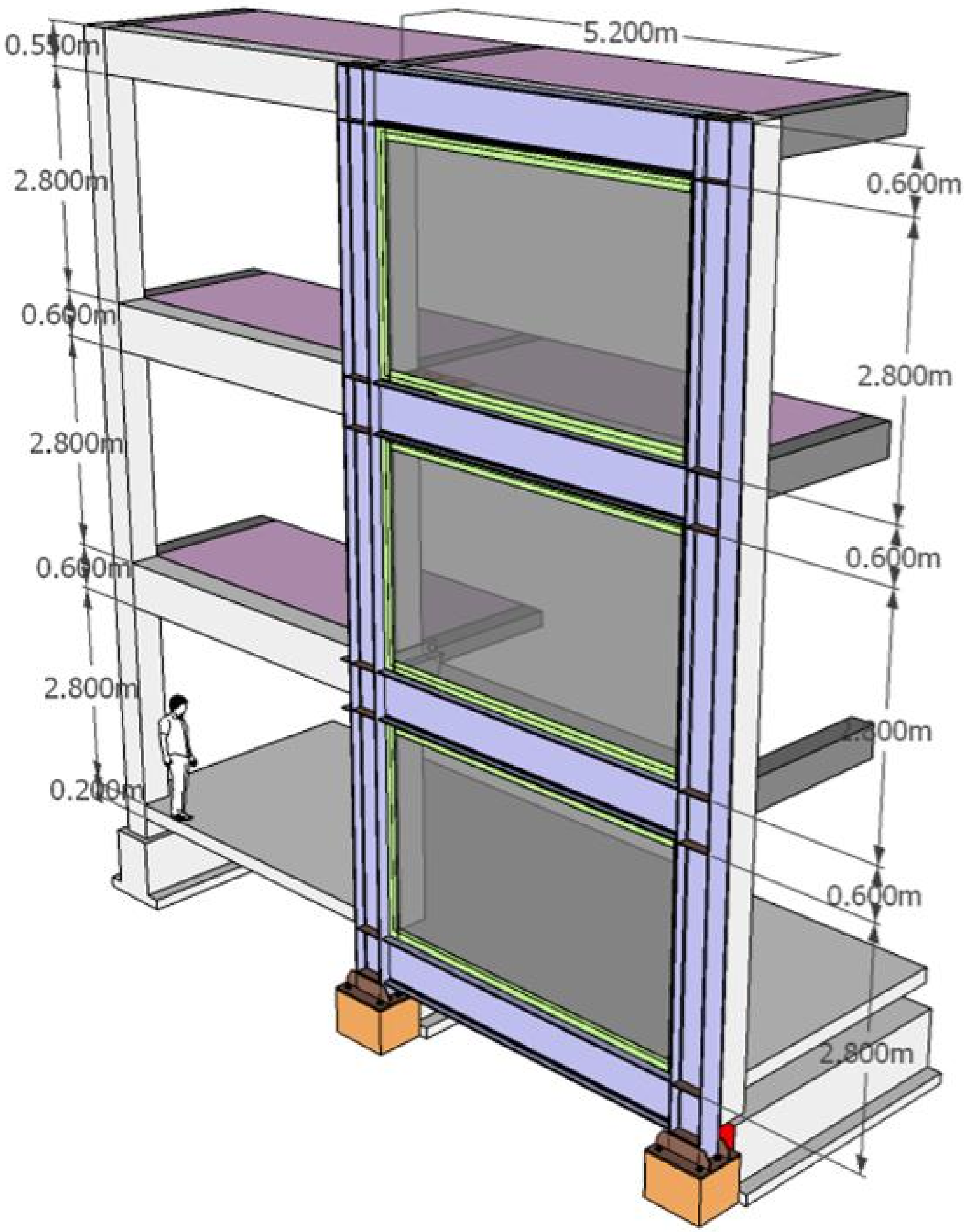

Figure 29. Isometric view of the proposed external shear wall solution.

Two alternative connection details proposed between the external frame and the concrete are presented in Figure 30 and Figure 31. The former solution relies on thick plates welded to the external frame and the plates are anchored to the concrete beams. The size of the parts must be verified by testing. The latter is based on L-profiles with additional welded plates. Again, the sizes of the parts are suggestive and should be analyzed and/or tested for real applications.

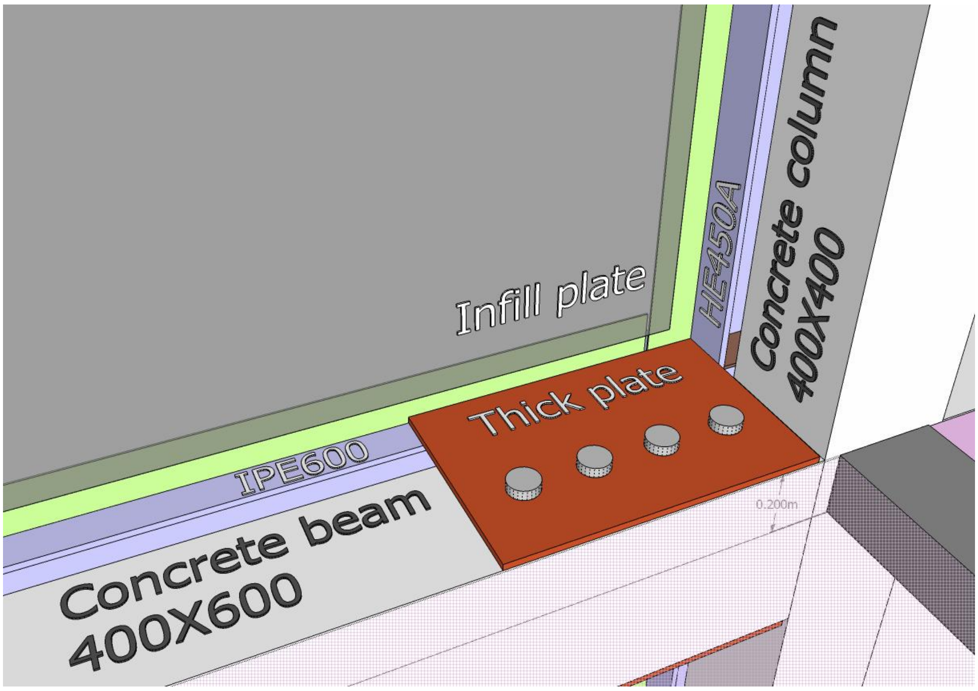

Figure 30. Proposed connection principle between the external frame and the concrete frame.

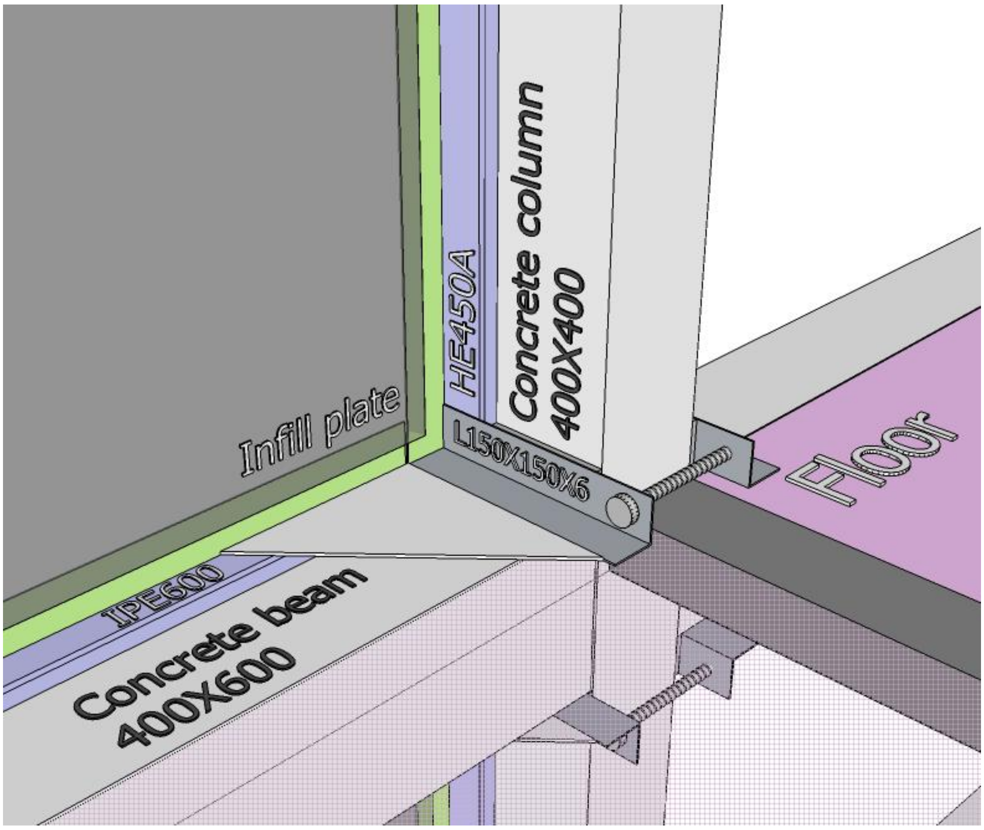

Figure 31. Proposed connection principle between the external frame and the concrete frame.

Figure 32 presents the proposed detail for the steel column foundation. Light brown refers to new concrete, which replaces also part of the original foundations.

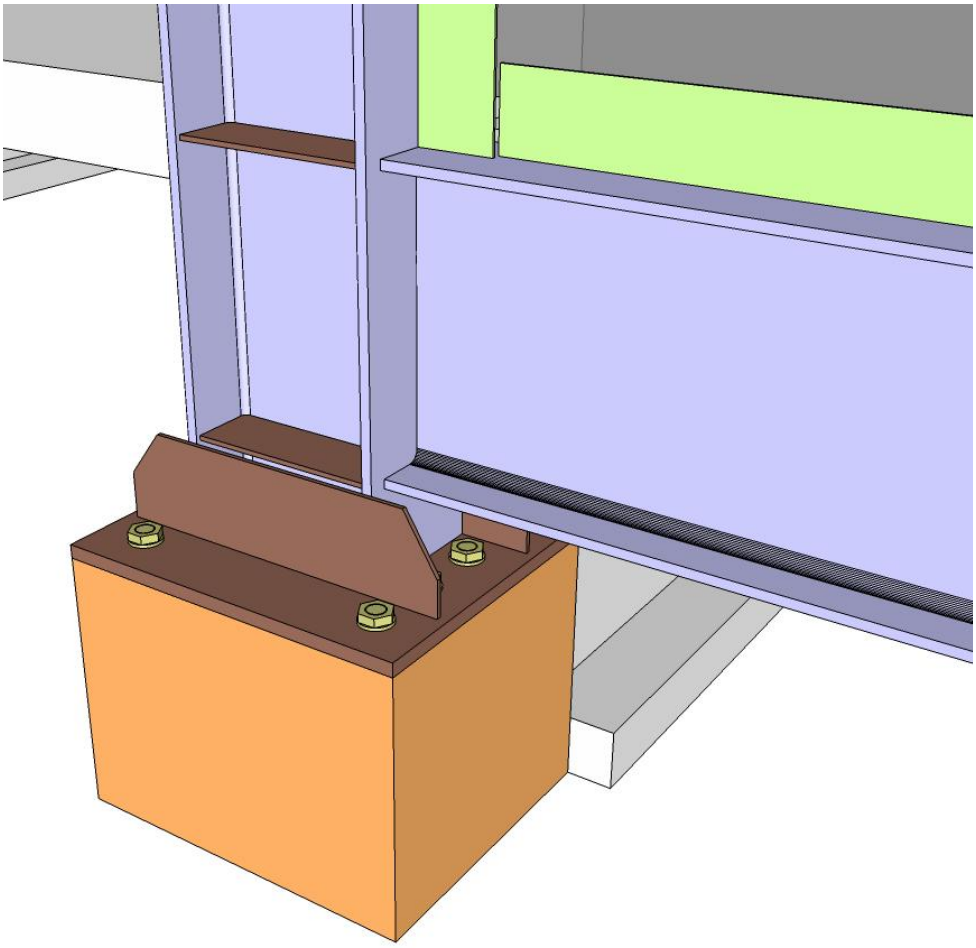

Figure 32. Proposed principle design of the steel column foundation.

One proposed solution for creating feasible retrofit solution is to replace the standard I-profile columns and beams with UPE-profiles, which is presented in Figure 33. This enables bringing the external frame even closer to the existing concrete structure.

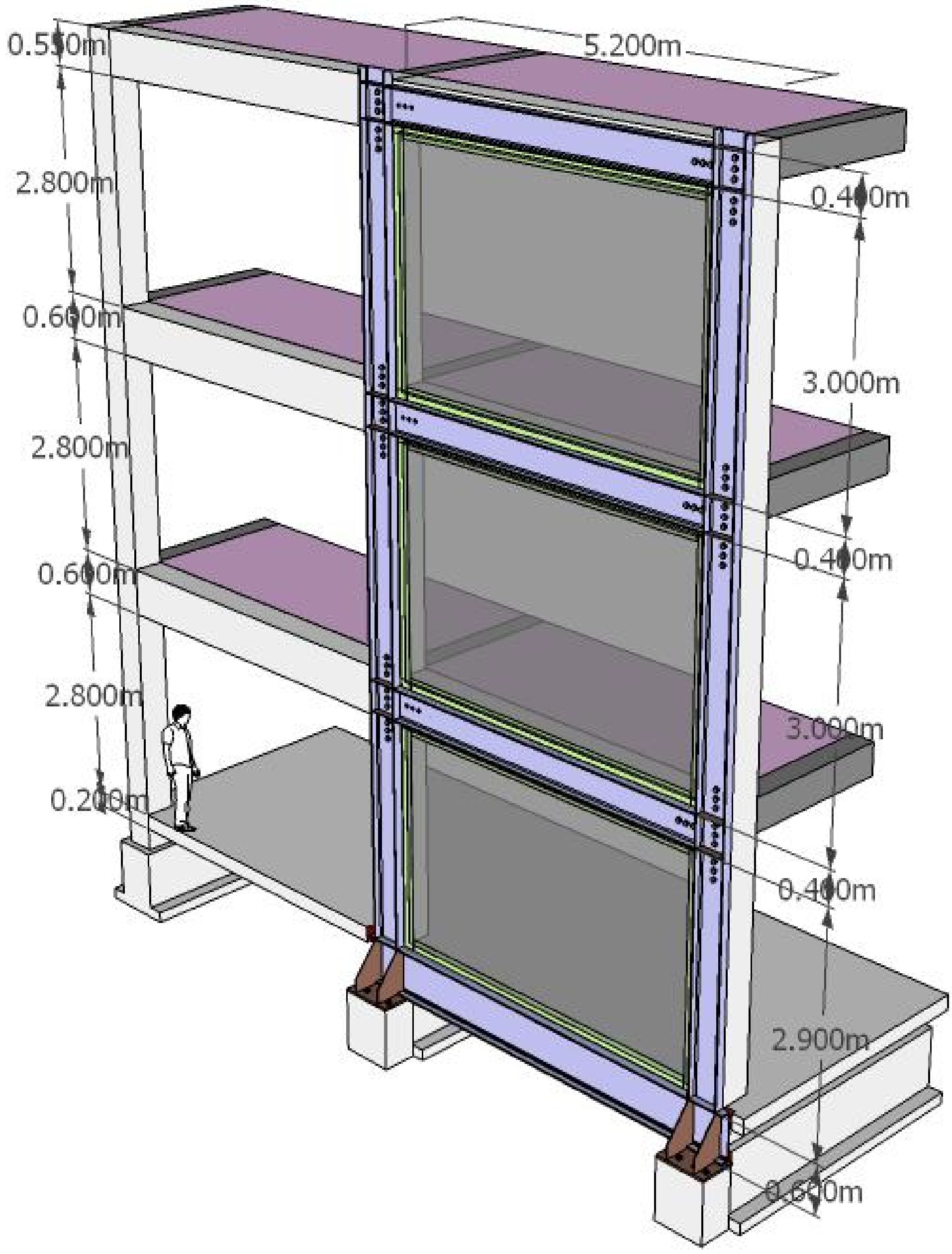

Figure 33. Isometric view of part of the building with the external shear wall.

The UPE-profiles can be connected to the existing concrete by using adhesive steel anchors as presented in Figure 34. However, the size and number of the anchors must be verified by testing. The existing concrete at the base must be demolished partly, to provide space for the IPE-profile forming a base for the UPE-columns. The amount of concrete demolishing can be reduced by cutting the flanges away from concrete side of the base beam (IPE). This should not largely affect the performance of the solution, but this way the column reinforcements might not be in the way. Light brown colour in Figure 34 refers to the new concrete foundation next to old one.

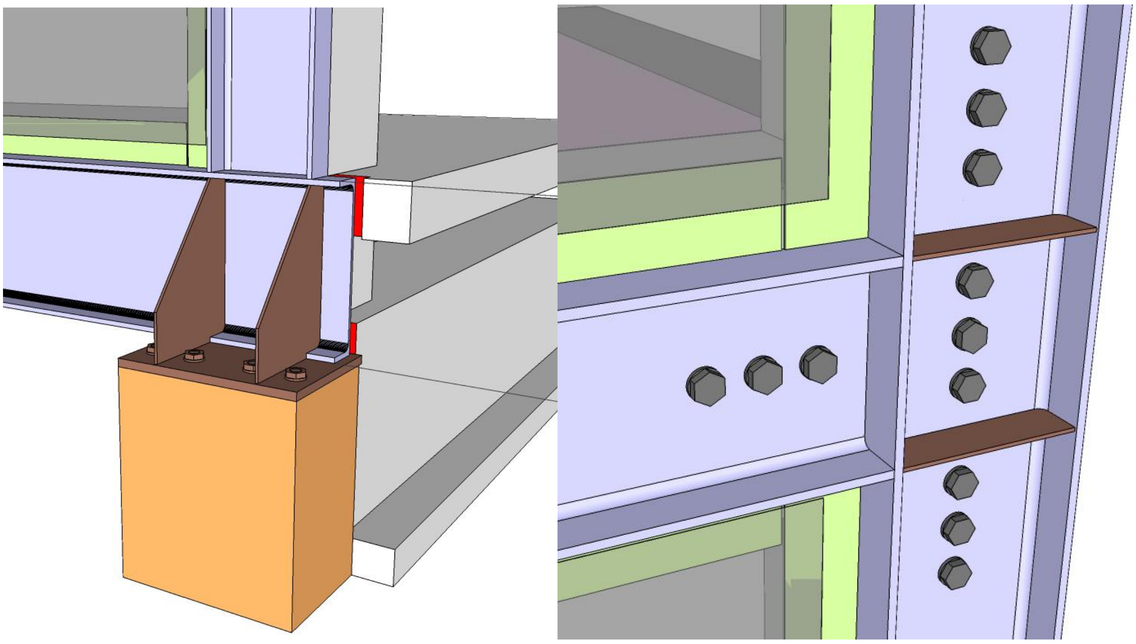

Figure 34. Base and beam-column connection principles for the external frame solution with UPE-profiles.

If the performance of anchoring cannot be verified, some of the anchors can be replaced by bolts through the concrete columns and beams as illustrated in Figure 35.

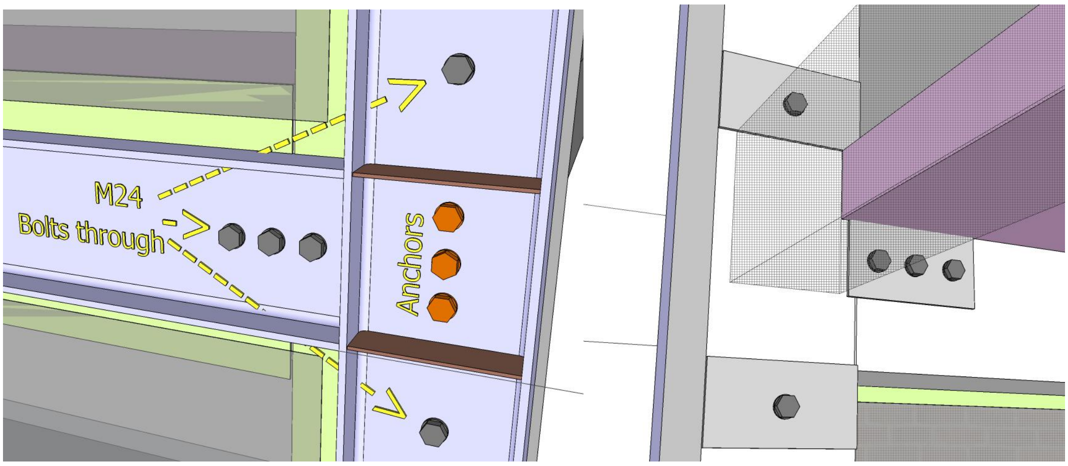

Figure 35. Combined anchor and bolted connection for UPE-frame.

4.2.1 Installation and assembly

Both external frame solutions, based on U- and I-profiles, can be manufactured at the machine shop, and then erected at the site, if the transportation of large frames is possible. Furthermore, both solutions need rework of the foundations near the concrete columns, but the advantage is on the U-profile solution, where the rework might be more minor. The clear advantage of the U-profile solution is that internal walls can remain intact when the frame is anchored to the concrete. The I-profile solution requires that at least parts of the walls must be taken down, in order that connection plates or profiles can pass through.

Figure 36 illustrates the possibility of installing lightweight “fake” brick wall façade to cover steel plate shear walls, which improves the outlook of the retrofitted building.

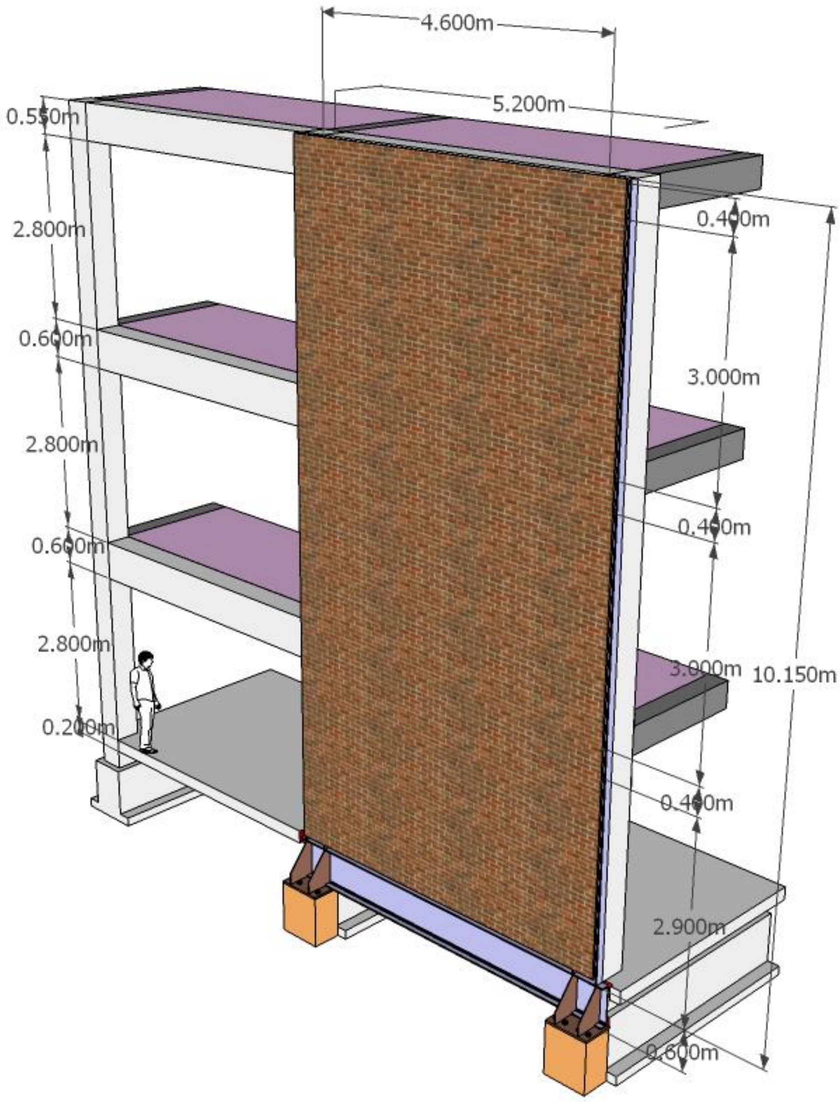

Figure 36.View of the brick wall installed for architectural considerations.

5 Conclusions

Steel plate shear walls have been proven to provide enhanced seismic performance to new and existing structures by increasing stiffness and ductility. However, most of the test results consider steel frames or only the shear wall, and steel plate shear walls connected to concrete frames are not studied. The challenges related SPSW retrofitted to work with concrete structures are related to the force transfer between these two structures. The case building proves to be difficult, because the concrete is weak making the connection detailing challenging.

A proposed solution for solving the problems with the case building using SPSW consists of an external frame manufactured using UPE-profiles and steel infill plates. This solution provides sufficient performance and a reasonable installation procedure. However, the anchoring of UPE-profiles to the concrete frame must be tested to verify the performance. The infill plate can be perforated, which reduces the requirements of the surrounding frame and the connections, while decreasing the amount of added stiffness and strength of the retrofit. Similar effect can be achieved also by using steel having a low yield point.

References

Driver, R.G., Kulak, G.L., Kennedy, D.J.L. & Elwi, A.E. 1998, "Cyclic Test of Four-Story Steel Plate Shear Wall", *Journal of Structural Engineering,* vol. 124, no. 2, pp. 112-120.

Fülöp, L. 2010, *LGS steel shear wall system to retrofit or upgrade r.c. frame structures*, VTT.

Li, J., Gong, J. & Wang, L. 2009, "Seismic behavior of corrosion-damaged reinforced concrete columns strengthened using combined carbon fiber-reinforced polymer and steel jacket", *Construction and Building Materials,* vol. 23, no. 7, pp. 2653-2663.

Mazzolani, F.M. 2008, "Innovative metal systems for seismic upgrading of RC structures", *Journal of Constructional Steel Research,* vol. 64, no. 7-8, pp. 882-895.

Montuori, R. & Piluso, V. 2009, "Reinforced concrete columns strengthened with angles and battens subjected to eccentric load", *Engineering Structures,* vol. 31, no. 2, pp. 539-550.

Ohmura, T., Hayashi, S., Kanata, K. & Fujimura, T. 2009, "Lateral strength evaluation of seismic retrofitted RC frame without adhesive anchors", *Excellence in Concrete Construction Through Innovation,* CRC Press/Balkema, London, UK, pp. 325.

Paulay, T. & Priestley, M.J.N. 1992, *Seismic design of reinforced concrete and masonry buildings*, .

Purba, R. & Bruneau, M. 2009, "Finite-Element Investigation and Design Recommendations for Perforated Steel Plate Shear Walls", *Journal of Structural Engineering,* vol. 135, no. 11, pp. 1367-1376.

Schumacher, A., Grondin, G.Y. & Kulak, G.L. 1997, *Connection of Infill Plates in Steel Plate Shear Walls*, University of Alberta, Edmonton, Alberta, Canada.

Shishkin, J.J., Driver, R.G. & Grondin, G.Y. 2009, "Analysis of Steel Plate Shear Walls Using the Modified Strip Model", *Journal of Structural Engineering,* vol. 135, no. 11, pp. 1357-1366.

Timler, P.A. & Gulak, G.L. 1983, "Experimental study of steel plate shear walls", *Structural Engineering Report,* vol. 114.