

**École polytechnique de Louvain**

# **Effect of floor diaphragms on the shear behaviour of RC walls in buildings**

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*"All models are wrong, but some are useful"*

- George E. P. Box

*"Dubium sapientiae initium. (Doubt is the origin of wisdom.)"*

- René Descartes

## **Abstract**

by Sami MESRAR

Lateral loads in buildings are often resisted by using reinforced concrete shear walls. Those loads are transmitted to the walls through the floor slabs of the structure, which act as diaphragms. While both are connected and load transfer takes place between the two, they generally are designed by themselves without considering the other.

However different interactions take place between those two elements which can impact the global behaviour of the structure. While the coupling of walls with beams is well documented and used in wall design, slabs are not considered rigid enough to act as a coupling element. However, different studies have shown that slab can indeed provide coupling to shear walls. Another, more unknown, interaction is one that is here referred to as "stirrup effect". This relates to the fact that the floor diaphragm, at its junction with the wall, is preventing expansion and cracks from opening.

This thesis is therefore looking to investigate those two interactions and to assess if they are indeed rightly ignored most of the time or if they should be considered more seriously.



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# Chapter 1

## Introduction

### 1.1 Motivation

Reinforced concrete shear walls are widely used around the globe in buildings to sustain lateral loads. Those loads, whether they are caused by wind or seismic actions, are transferred to the walls by the floor diaphragms of the structure. The floor slabs which also mainly serve as support for the vertical loads such as the users of the building or the various objects, therefore, have a certain role to play in the lateral load resisting system.

However, those diaphragms are often overlooked during the analysis of a structure. They are most of the time considered rigid for the horizontal load distribution between the different walls but they then are considered not rigid enough to exert a coupling effect by linking the different walls together.

The load distribution and coupling are two of the main interactions floor diaphragm can have with RC shear walls. Some researchers have already established the fact that those interactions can have a significant effect on the behaviour of shear walls and thus on the structure itself.

Another interaction is generally not considered in regular structural analysis. A floor diaphragm, at its joint with the wall, can prevent the latter from expanding in its plane and therefore prevent cracks from opening at this very location. Such an action could potentially affect the shear behaviour of the reinforced concrete wall and is denoted in this thesis as the "stirrup effect" since the floor slab would act sort of like a discrete stirrup.

This work, therefore, aims at investigating this still relatively unknown stirrup effect as well as the coupling of walls through floor slabs.

### 1.2 Objectives

Using mainly a finite element software - Vector2 - for the analysis of reinforced concrete structures, this thesis aims at evaluating the possible impact of the previously mentioned

interaction on the global behaviour of the walls. The resistance of the wall will be studied as well as its displacement capacity, which is one of the main attributes one can seek for an RC shear wall.

## 1.3 Outline

The second chapter will present the main aspects behind shear walls behaviour. The different failure modes as well as an introduction to the seismic design approach of the Eurocode also figure in this chapter. Furthermore, different types of wall-diaphragm interactions are also presented.

Chapter 3 goes over the finite element software features as well as different modelling choices.

In the next chapter, the selected model will be tested with existing laboratory test data to validate it and prove its ability to capture the behaviour of RC shear walls.

The fifth chapter presents an extensive parametric analysis of different wall configurations which are intended to highlight the participation of the floor diaphragm in the wall behaviour via the "stirrup effect".

Chapter 6 is dedicated to the coupling effect and its impact on the performance of associated shear walls.

Finally, the conclusions and outlook for further research are presented in the seventh chapter.

## Chapter 2

# Theoretical background

Due to winds and earthquakes, buildings can be subject to horizontal forces. The higher the building, the more complicated it is to deal with such types of loads since they can cause significant efforts, especially at the base of the structure, as well as undesirable lateral displacements and deformations. In order to resist those solicitations, different structural solutions exist and have each their strengths and weaknesses. Regarding reinforced concrete constructions, rigid frames, shear walls, or a combination of the two are the most used systems.

This thesis is mainly focused on reinforced concrete shear walls, more precisely the influence a reinforced concrete floor diaphragm can have on the wall's behaviour. This first chapter then presents the theoretical background needed for the understanding of this topic, as well as a review of the relevant existing literature and laboratory tests.

### 2.1 Shear walls

In buildings where structural walls are present, they are usually used as bracing only and the gravity loads they bear are rarely the limiting factor of their design. Other gravity loads resisting systems such as columns are often used in combination with those walls and, unless it has been engineered as a dual system, are not accounted for in the lateral-load-resisting system (LLRS). Reinforced concrete shear walls are then designed apart from the rest of the structure, as will be discussed in Section 2.2.

Thanks to their high in-plane stiffness, structural walls provide buildings braced by this system with in-plane resistance to lateral displacement and allow them to deform elastically under wind loads. For low to mid-rise buildings, shear walls are not mandatory structural elements, but in order to reach heights of multiple tens of floors, it is necessary to use them for better deflection control and economic efficiency.

In the case of an earthquake, which can cause displacements significantly more important than those caused by wind, designing a structure behaving elastically would result in an economically inefficient solution as it would require a very high yield resistance from the wall. Capacity based design, which will be discussed in Section 2.2 allows permanent

deformation to take place in a defined way and location. This way, the performance of the structure under seismic solicitation can be better controlled, and it allows for a ductile behaviour.

The shear wall systems, therefore, present the following advantages and disadvantages compared to frame systems, also listed by M. Fardis in [1]:

- ⊕ Walls are stiff, they prevent or limit the damage from seismic actions.
- ⊕ Soft-story mechanism can not develop (except if weak connection to foundations)
- ⊕ Lower quality design or poor site implementation has less impact on individual walls than on frames
- ⊕ Walls are more cost-effective
- ⊖ Less ductile and more vulnerable to shear
- ⊖ Lower redundancy, fewer alternative load paths
- ⊖ Limited architectural manoeuvrability
- ⊖ Not cost-effective to sustain vertical loads
- ⊖ State-of-the-Art on cyclic behaviour of walls is less advanced than of frames since experiments, as well as analytical and numerical studies, are more challenging. Walls are more difficult to accurately model and design, there is, therefore, more uncertainty around their behaviour in seismic conditions.

### 2.1.1 Behaviour and failure modes

When external horizontal loads are applied to the building and then distributed to the different walls given the geometry and the stiffness of each of them, the wall can be represented as a cantilever beam embedded in the foundation. The bending moment and curvature follow the relationship as shown on Figure 2.1. The more slender the wall the more it will behave like an actual cantilever beam and on the contrary if it is less slender, other phenomenons will occur as it will be discussed in a later section.

As the main topic of this thesis is the influence of the floor diaphragms, it is presumed that those effects take place under large displacement and deformation of the wall, which is often the case during earthquakes. Walls which have not been designed to resist those seismic actions will therefore suffer major damage and can cause the structure they belong

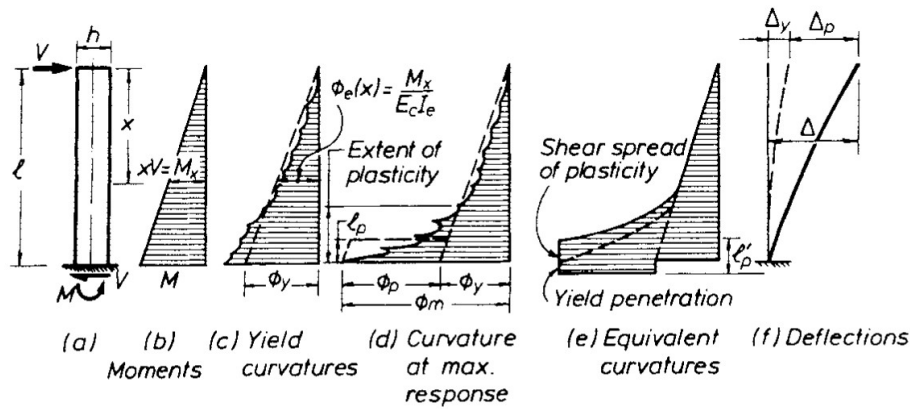


Figure 2.1: Relationships for prismatic reinforced concrete wall: (b) moments, (c) yield curvature, (d) curvature at max response, (e) equivalent curvature, (f) deflections. Source : [2]

to, in some cases, to collapse. Depending on a multitude of parameters, the wall will deform, experience cracking and ultimately fail following different mechanisms displayed in Figure 2.2.

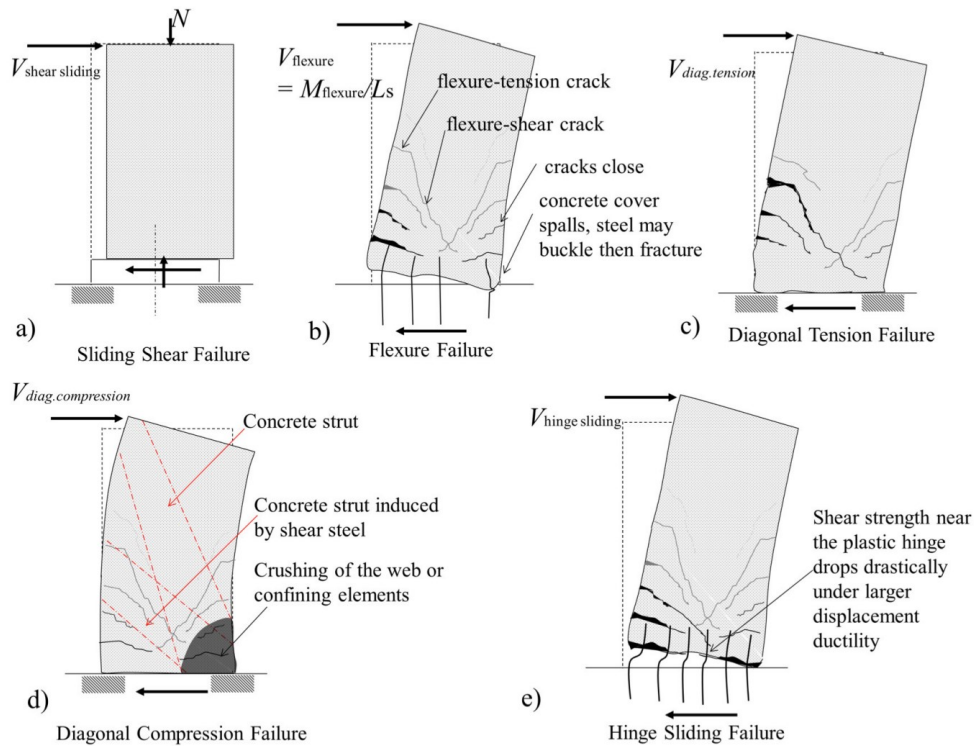


Figure 2.2: Typical failure mechanisms of RC walls: (a) sliding shear, (b) flexure, (c) diagonal tension, (d) diagonal compression (crushing of web and/or boundary elements), and (e) hinge sliding. Source : [2]

Some of those failure mechanisms are more desirable than others. As ductility is a key criterion for the seismic design of structures, the deformation and energy dissipation that the occurring failure mechanism allows taking place are extremely important in its suitability. If a certain failure mode is reached before a sufficient amount of energy has been dissipated, then it is to be proscribed in favour of a more ductile one. The main

source of energy dissipation for a reinforced concrete wall resides in the yielding of its longitudinal steel reinforcements in tension. Therefore, if any failure occurs before the yielding of the longitudinal reinforcements, it is to be avoided as there is a lack of ductility.

Three main types of failure can then be defined:

1. Flexural failure
2. Shear failure
3. Combined shear-flexure failure

#### 2.1.1.1 Flexural failure

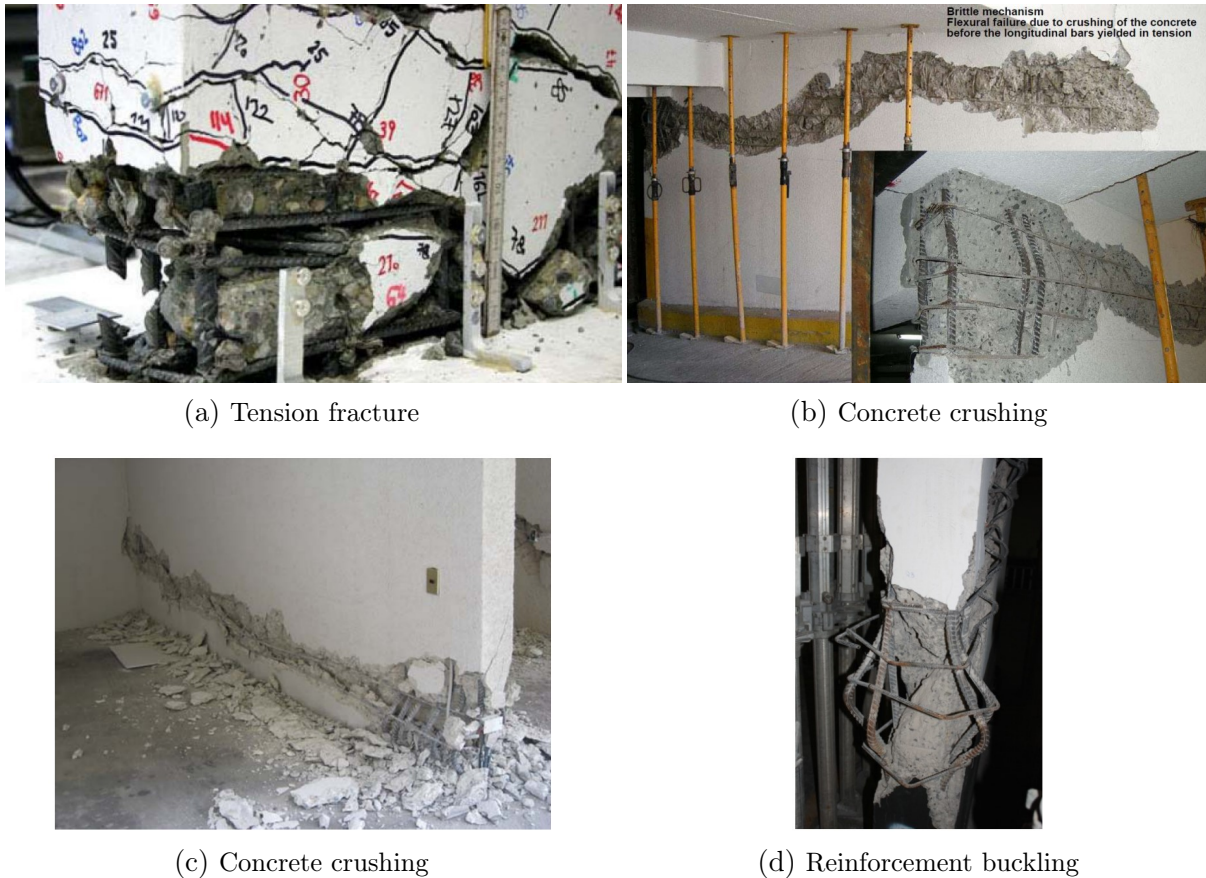


Figure 2.3: Examples of real flexural failures. (a) and (b) source: [3], (c) source: [4] (d) source: [4]

Flexural failure, of which real life example are shown in Figure 2.4, regroups all the mechanisms inherent to a flexural behaviour without any shear considerations, namely:

- Tension fracture of longitudinal reinforcements (at large strains)

- Crushing of concrete
- Buckling and fracture of longitudinal reinforcements (after the yielding of the tension reinforcements)

The first two allow for the yielding of the tension reinforcement to occur and therefore can be considered ductile as energy is dissipated, and larger deformations occur before the failure. Concrete crushing can, depending on the design, take place at different times. The compressed concrete can fail before or after the yielding of the longitudinal reinforcements and before or after the buckling of compression reinforcements.

### 2.1.1.2 Pre-emptive shear failure

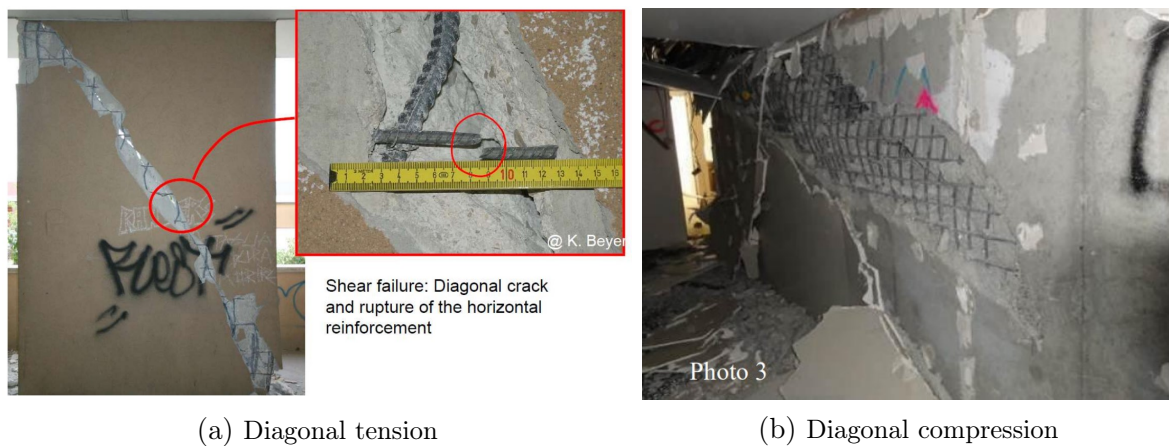


Figure 2.4: Shear failures, real life examples. (a) source: [3], (a) source: [5]

Shear failures are in general considered as brittle failure. Their occurrence is not directly linked with the yielding of the longitudinal reinforcements. They can therefore cause a sudden drop of lateral load resistance before the dissipation of energy and are thus proscribed by many standards which provide measure to prevent those types of failures. Three different mechanisms are possible:

- Diagonal tension failure: fracture of transverse reinforcement in tension
- Diagonal compression failure: rupture of the diagonal concrete strut
- Sliding shear or shear friction failure

### 2.1.1.3 Flexure-shear failure

Flexure-shear failure are all the failure for which the wall has shown a behaviour linked to both shear and flexural mechanisms. Ductile shear failure as well as flexure failure exhibiting large diagonal cracks are considered in this failure type. These combined failures can have variable ductility depending on the detailing of the wall.

## 2.1.2 Factors influencing the behaviour

The behaviour and failure modes described in the previous section of course depend on different parameters. Exterior parameters such as axial loading, geometrical parameters such as aspect ratio of the wall, materials properties or design parameters such as the detailing and reinforcement ratios all influence the behaviour and the performance of shear walls.

### 2.1.2.1 Axial loading

In general, shear walls are not part of the principal gravity load bearing system, as their main focus is on lateral load resistance. However, they still are subject to axial loading, starting with their own weight. This parameter is denoted by the axial load ratio (ALR):

$$\text{ALR} = \frac{N}{A_c f_c} \quad (2.1)$$

This ratio is usually inferior to 30%, and it plays a certain role in the behaviour of shear walls.

The more a shear wall is axially loaded, the higher will be its horizontal load bearing capacity [6]. It will on another hand reduce its horizontal displacement and its plastic rotation. High axial load ratios can also lead to out-of-plane wall deformations and instability, especially for thinner walls, as their weaker inertia is lower. Even if floor diaphragm obviously have an influence on this phenomenon as they maintain the wall in its plane at the intersection, this topic is not part of the presented thesis.

### 2.1.2.2 Aspect ratio

The aspect ratio of a wall is defined by the ratio between the base bending moment  $M_b$ , the base shear  $V_b$  and the wall's section depth  $l_w$ . It can also be expressed as the ratio between the shear span  $L_s$  and the wall length.

$$\text{AR} = \frac{M_b}{V_b l_w} = \frac{L_s}{l_w} \quad (2.2)$$

In the case of laboratory push-over tests on walls, which are often conducted by applying a single horizontal force at the top of the specimen, the aspect ratio can be defined as the ratio between the height of the wall and its depth.

Aspect ratio is one of the key factor which can influence the failure type of a reinforced concrete wall. With an aspect ratio superior to 3, the wall is considered slender and is supposed to exhibit a flexure dominated behaviour. Squat walls are then defined by an aspect ratio inferior to 3. Walls with an aspect ratio below 1.5 present shear-controlled behaviour while. An aspect ratio between 1.5 and 3 is therefore synonym of a behaviour influenced by both shear and flexure [7].

## 2.2 Design approach of the Eurocode

### 2.2.1 General principle

Before any standards recommendations, the phase where the designer makes important decisions and can best showcase its creativity and skills is significant in the later stages of the design procedure. The so-called conceptual design is then a decisive phase in every project and the fundamental attributes, discussed in [1], include among others:

- A clear lateral load resisting system (LLRS)
- Simplicity and uniformity of the system
- Redundancy of the system

Following those ideas will ease the remaining of the design process. Paired with quality analysis and detailing, a good conceptual design will ensure an adequate behaviour of the structure in the case of a seismic event.

As explained previously, seismic events can induce severe deformations to buildings and create important forces in the LLRS, often leading to failure and collapse of the vulnerable structures. To limit the physical damages and therefore the economical liabilities caused by an earthquake, it is important to design a structure with those considerations in mind and sufficient resistance and displacement capacity need to be ensured.

Most of the currently used seismic standards are centred around a capacity-based approach to the seismic behaviour of structures. This allows the designer to specifically choose which element will ensure the ductility of the structure. Figure 2.5 presents an analogy of the capacity-based design with a chain including a ductile link which guides the behaviour and deformation capacity of the chain. As the other elements are well designed and can sustain heavier forces, the ductile element allows the structure to deform before failing as it focuses most of the strain.

Designing an element subjected to imposed displacement with a ductile behaviour can prove very cost-effective when compared to an element supposed to behave elastically. Given it can dissipate energy, its ultimate resistance can be substantially lower than the elastic element, which can drastically reduce its required dimensions and make it less expensive. The Eurocode will then allow the designer to use a reduction factor -  $q$ , which will be discussed later - on the equivalent forces in order to design specific elements to withstand a lower load.

To this end, EN1998-1: 2004 prescribes three different analysis methods :

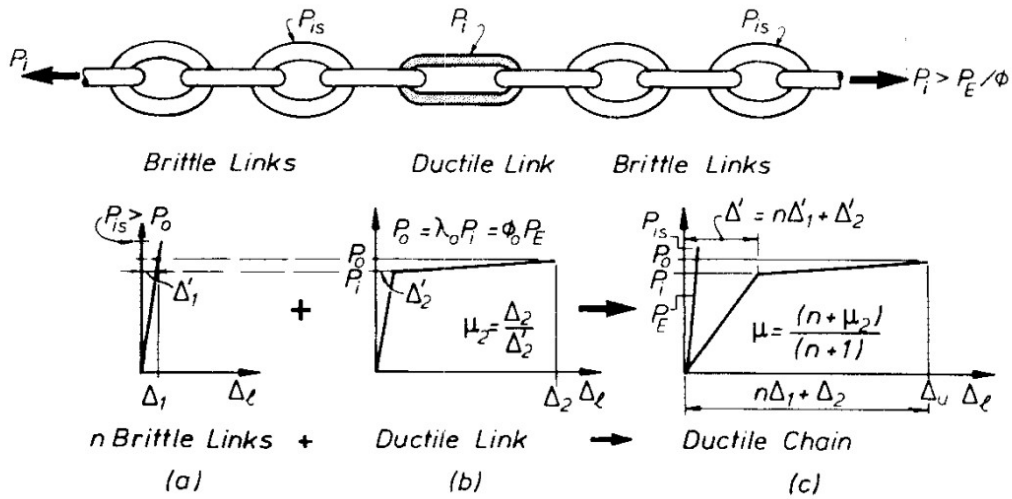


Figure 2.5: Principle of capacity design and strength limitation illustrated by a chain. Source : [8]

- Lateral forces method of analysis
- Modal response spectrum analysis
- Non-linear methods linear analysis

### 2.2.2 Equivalent lateral forces method (ELFM)

This method, being the simplest, can only be used under certain conditions, the main one being that the behaviour of the structure is not significantly affected by the other modes than the first one.

The base shear is computed as follows :

$$F_b = S_d(T_1) \cdot m \cdot \lambda \quad (2.3)$$

In which :

- $S_d$  is the design spectra value for period  $T_1$  - see next paragraph
- $T_1$  is the fundamental period of the building in the considered direction
- $m$  is the total mass of the building
- $\lambda$  is a correction factor

It is worth noting that EC1998-1: 2004 offers different approximation formulas for  $T_1$  even if they are quite conservative as they underestimate this period.

### 2.2.2.1 Response spectra

As it is impossible to predict the intensity of a future earthquake, a reference elastic spectrum was statistically established to cover a wide range of possible seismic events. The Eurocode actually prescribes two types of spectrum, depending mainly on the soil type from which the envelope is to be considered. The design elastic spectrum is obtained by multiplying the reference spectrum by the peak ground acceleration (PGA) which should be specified regionally in the different national appendices.

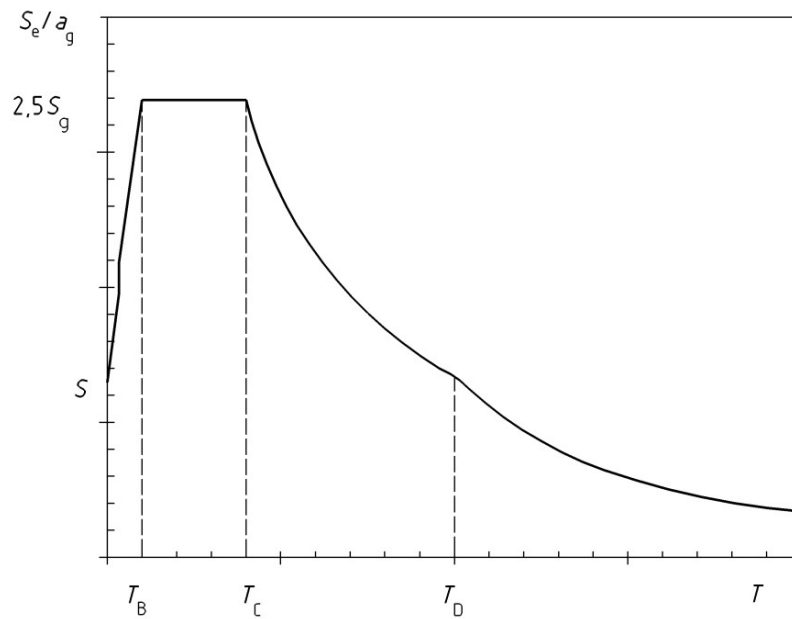


Figure 2.6: Reference elastic spectrum. Source : [9]

This reference elastic spectrum is ruled by the following expressions :

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[ 1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1) \right]$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5$$

$$T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_C}{T_D}$$

$$T_D \leq T \leq 4s : S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_C T_D}{T^2}$$

$S$ ,  $T_B$ ,  $T_C$ , and  $T_D$  depend on the soil type,  $\eta$  depends on the damping and  $a_g$  refers to the PGA and the importance factor.

The design response spectrum is then directly derived from the previous elastic one using the “behaviour factor”  $q$ . EC1988-1: 2004 defines this factor as follows:

*The behaviour factor  $q$  is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5 % viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure*

This factor is then used to reduce the value of the elastic response spectra and design a structure taking into account its energy dissipation. It allows the designer to not oversize the structure as the ELFM is based on an elastic design as its value generally ranges from 1.5 to 2.

### 2.2.2.2 Forces distribution

The distribution of the horizontal forces can be described as follows :

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \quad (2.4)$$

where

- $F_i$  is the horizontal force acting on storey  $i$
- $F_b$  is the seismic base shear from Eq. 2.3
- $m_j, m_j$  are the storeys masses
- $s_j, s_j$  are the displacements of masses  $m_j, m_j$  in the fundamental mode shape;

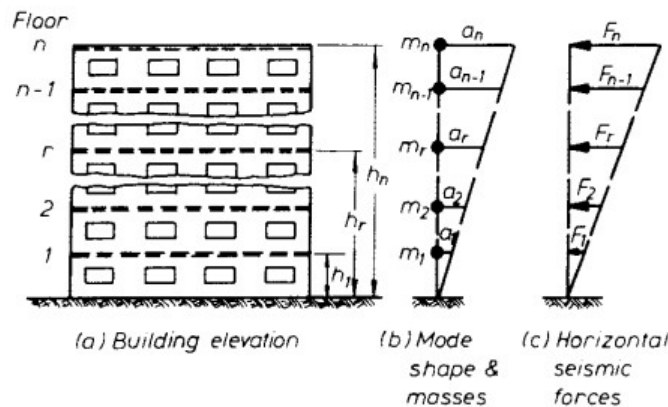


Figure 2.7: ELFM for base shear distribution. Source : [8]

The distribution can also be approximated by a linear distribution. In which case,  $s_j$  and  $s_j$  can be replaced by  $z_j$  and  $z_j$ , the heights of the masses  $m_j$ ,  $m_j$  above the level of application of the seismic action.

If the conditions allowing to use the ELFM are not met, the response of all modes of vibration contributing significantly to the global response shall be taken into account. The combination of modal responses will then define the base shear  $s$  as well as all the other seismic action effects.

### 2.2.3 Non-linear methods

In contrast to usual solicitations, seismic action imposes displacements on a structure rather than forces. The previously presented ELFM could therefore seem inappropriate, and a non-linear static - or "pushover" analysis seems more fitting.

This analysis allows considering the ductility of the LLRS and aims to find a performance point satisfying the different design requirements. Structure is to be first simplified into an equivalent SDOF from which a capacity curve is derived using a pushover analysis. This curve is then idealized with an elastic-perfectly plastic nonlinear force-displacement relationship.

This capacity curve is then to be matched with the demand curve derived from the elastic response spectra described earlier using the equal displacement and equal energy principle. The following figure illustrates those two principles and how they allow for the calculation of the displacement demand  $d_t$  and therefore the ductility factor  $u_\delta = d_t/d_y$  ( $d_y$  being the yield displacement).

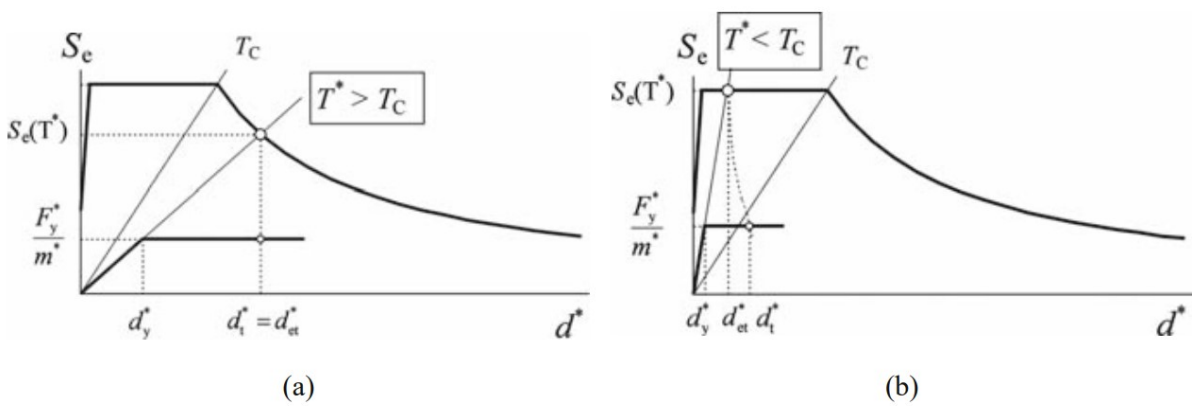


Figure 2.8: Illustration of a) equal displacement principle, b) equal energy principle. Source : [1]

$$q = m^* S_a(T^*) / F_y^* \quad (2.5)$$

$$\text{if } T < T_C : u_\delta = 1 + (q - 1) \frac{T_C}{T} \quad \text{and} \quad q = 1 + (u_\delta - 1) \frac{T}{T_C} \quad (2.6)$$

$$\text{if } T > T_C : u_\delta = q \quad (2.7)$$

With the target displacement  $d_t$  limit states can be verified both at global and local levels. By considering the non-linearity of the structure and its ability to dissipate energy through permanent deformation, these methods allows designing a structure to forces which are way lower than in the case of a purely elastic design. The behaviour factor which translates this reduction can take values from 3 to 4 (even 5) [3].

### 2.2.4 Specific design prescriptions

As explained in Section 2.2.1, the Eurocode aims at a certain ductility of the structure it helps designing. The code defines as critical regions the zone in which the energy dissipation and therefore most of the permanent deformation occurs. For shear walls systems, the critical regions are of course located at the base of the wall, where the plastic hinges are formed. A wide range of different studies have been conducted to find relatively easy ways to estimate this plastic hinge length. Eurocode 8 proposes different values from the simplest and most conservative :

$$L_{pl} = \max[l_w, H_w/6] \quad (2.8)$$

To more specific ones, in EN1998-3 for example:

$$L_{pl} = 0.1L_s + 0.17l_w + 0.24 \frac{d_{bL} f_y}{\sqrt{f_c}} \quad (2.9)$$

Where:

$L_s$  is the shear span  $M_b/V_b$

$l_w$  is the depth of the wall section

$d_{bL}$  is the (mean) diameter of longitudinal rebars in the plastic zone

$f_y$  is the yield strength of the reinforcements

$f_c$  is the compressive strength of concrete.

For walls, the value of  $L_{pl}$  from equation 2.9 ranges from 0.18 to 0.24  $L_s$ . In this region

where energy dissipation takes place in the form of relatively important curvature ductility and strain, special detailing rules must apply to insure the capacity of the wall to deform sufficiently. Those detailing requirements depend on the ductility class of the building, and they are stricter with the increasing ductility demand.

### 2.2.4.1 Confined concrete

As flexural behaviour is the only reliable mechanism allowing for controlled energy dissipation in walls, wall sections must be accordingly designed and detailed. Like EN1992, EN1998 takes into account the increased properties of concrete when it is confined by transverse reinforcement hoops. In a wall, the boundary concrete is allowed to spall as it reach its ultimate strain as the concrete in between the confining reinforcement is reaching its enhanced ultimate strain as illustrated in Figure 2.9.

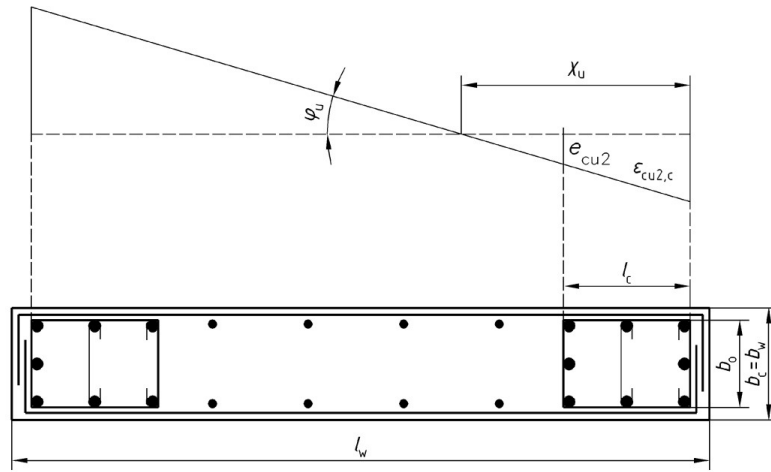


Figure 2.9: Strain distribution of confined boundary zone in a wall at ultimate curvature. Source : [9]

EN1992-1 proposes the following ultimate strain value for confined concrete:

$$\epsilon_{cu2,c} = \epsilon_{cu2} + 0.2\sigma_2 f_{ck} \quad (2.10)$$

Where :

$\epsilon_{cu2}$  is the ultimate strain of unconfined concrete

$\sigma_2$  the effective ultimate lateral pressure due to the confining reinforcements, which can be taken as  $\sigma_2 = 0.5\rho_w f_{yw}$

Multiple studies have been led to find the best fitting ultimate strain of confined concrete, among them, Paulay and Priestley have proposed the following:

$$\epsilon_{cu2,c} = 0.004 + 2.8\epsilon_{su,w} \frac{\sigma_2}{f_{c,confined}} \quad (2.11)$$

EN1998-3 has put forward yet another formula for this ultimate strain, which makes small adjustment to the one proposed in EN1992 :

$$\epsilon_{cu2,c} = 0.004 + 0.5 \frac{\sigma_2}{f_{c,confined}} \quad (2.12)$$

#### 2.2.4.2 Preventing pre-emptive shear failures

Shear verifications in Eurocode 8 are done using the same equations as in Eurocode 2. The shear resistance of the wall being controlled by the diagonal compressive strength of the concrete  $V_{Rd,max}$  and the transverse reinforcement resistance  $V_{Rd,s}$ .

For structure belonging to a high ductility class (DC H), the shear resistance in case of a diagonal compressive failure of the concrete in the web should be taken as 40% of the one computed using EN1992 equations.

In the inelastic domain, the maximum shear in the wall is dictated by its flexural strength. The whole flexural plastic mechanism can only be achieved if the shear strength of the member is sufficient to allow a plastic hinge to form. However, having a sufficient elastic shear capacity may prove not to be enough to resist shear in the inelastic domain. It has been observed that shear resistance tends to degrade when submitted to cyclic inelastic displacements [10]. Therefore, shear failure can occur in this inelastic range even without an increase of shear demand. Force-displacement curves then represent this shear demand depending on the deformations of the wall and can be used to detect a failure caused by shear. EN1998-3 prescribes the following equation to estimate this shear strength degradation :

$$V_R = \frac{1}{\gamma_{el}} \left[ \frac{h-x}{2L_v} \min(N; 0.55A_c f_c) + \left( 1 + 0.05 \min(5; \mu_{\Delta}^{pl}) \right) \cdot \left[ 0.16 \max(0.5; 100\rho_{tot}) \left( 1 - 0.16 \min \left( 5; \frac{L_v}{h} \right) \right) \sqrt{f_c} A_c + V_w \right] \right] \quad (2.13)$$

Where:

$\gamma_{el}$  is 1.15 for primary elements and 1 for secondary seismic elements

$h$  is the depth of cross-section

$x$  is the compressive zone depth

$N$  is the compressive axial force

$L_v$  is the shear span ratio

$A_c$  is the cross-section area

$f_c$  is the concrete compressive strength

$\rho_{tot}$  is the total longitudinal reinforcement ratio

$V_w$  is the contribution of transverse reinforcements  $\rho_w b_w z f_{yw}$

This  $V_R$  value can not be taken as superior to the resistance of the web's concrete crushing, also described in [11]. Different equation have been proposed in the literature, some of which have been analysed in [10]. However, studies have shown that compared to tested specimen, those equations tend to underestimate the shear resistance [12].

Detailing requirement of Eurocode 2 already impose minimal quantities for the horizontal reinforcements as part of the general shear behaviour of walls, and EN1998.1: 2004 uses the same values. For low and medium ductility classes, there should be a minimum transverse reinforcement ratio in the web of the greatest of 0.1% and 25% of  $\rho_l$  with a maximal spacing of 400 mm. For high ductility classes structures, transverse reinforcement ratio in the web of the wall should be at least 0.2%, with rebars with diameter of more than 8 mm and with a maximal spacing of  $\min(25d_h, 250 \text{ mm})$ .

## 2.3 Floor diaphragm

Floors have the primary function to of course create usable space in a building by carrying vertical loads over a large surface and transferring them to the vertical load system. But here is not their only function, they also serve as a mean to transfer the horizontal loads coming from the facades (ex: winds) to the LLRS. From a seismic point of view they can transmit the inertial forces of the masses, mainly concentrated in the floor themselves, to the LLRS.

## 2.4 Wall-diaphragm interactions

As briefly explained in section 2.2, shear walls are designed as a self-standing cantilever beam without really considering its connection to the rest of the structure (unless it is incorporated in a mixed LLRS). Nonetheless, the floor diaphragm to which it is linked have different effects on the wall behaviour. They have not been widely covered in the existing literature, as conservative assumptions already ensure a quite safe design of walls and their impact may not be significant. The three main interactions are described in the remaining of this chapter, and the last two actually constitute the main focus of this thesis.

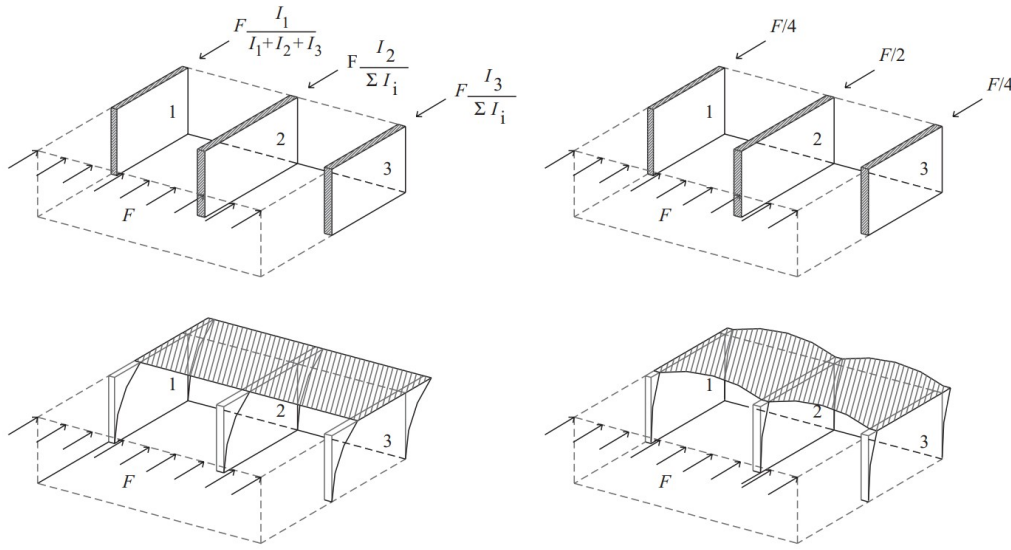


Figure 2.10: Diaphragm behaviour under horizontal loads: top) shear distribution, bottom) lateral displacement ; left) rigid diaphragm, right) flexible diaphragm. Source : [13]

### 2.4.1 Load distribution

In order to evenly distribute the forces to the different LLRS elements, the diaphragm is to remain elastic and to deform as little as possible. It then has to be both stiff and have a high enough strength to prevent deformation and if possible cracking. It so, it can be considered rigid and therefore modelled as a node with 3 DoF : 2 translation DoF in its own plane and a rotation DoF around the normal to its plane. This covers most cases and this assumption of rigid diaphragm is widely used in the design practice, but some configurations can cause the diaphragm to behave in a more flexible way.

A flexible diaphragm will deform in its own plane, which can cause different effects on both the diaphragm and the LLRS. If the floor plan has a large aspect ratio or is in a U, L, or T shape for example and does not present a seismic joint, high flexural deformations and stress could be observed.

With a deforming diaphragm, the distribution of horizontal forces between the different elements of the LLRS is also altered as it does not only depend on the horizontal stiffness of each element but also on the in-plane stiffness of the diaphragm like illustrated in Figure 2.10.

Different studies [14], [15] have demonstrated that the excess in shear loading of the walls can in certain case be significant (up to 40%) when considering a flexible diaphragm. If distribution of lateral forces to the LLRS elements deviates from the flexible to the rigid model, the flexible one must be considered. [16] have proposed multiple threshold above which flexibility should be taken into account. According to EN1998-1: 2004 :

*The diaphragm is taken as being rigid, if, when it is modelled with its actual in-plane flexibility, its horizontal displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements in the seismic design situation*

This particular interaction between the floor diaphragms and the LLRS has not been extensively investigated in the literature, but as it is not part of the scope of this thesis, it will not be discussed further.

### 2.4.2 Coupling effect

The coupling of adjacent shear walls through a coupling beam is something which has already been covered to a certain extent by the existing literature. Eurocode 8 defines a coupled wall as a:

*Structural element composed of two or more single walls, connected in a regular pattern by adequately ductile beams ("coupling beams"), able to **reduce by at least 25% the sum of the base bending moments** of the individual walls if working separately*

A wall with regular openings along its height could then be considered as actually two coupled walls, linked by multiple coupling beams between those openings. The difference in behaviour between the two systems, one where the walls are independent and the one in which they are interconnected, is significant as shown in Figure 2.11. These systems are particularly efficient to obtain a ductile behaviour, since they apply deformation by rotation to the beams and bring them to their yielding point. Those coupling elements can thus dissipate energy if they have been properly designed and detailed able to dissipate large amount of energy.

The coupling of the walls will influence their axial load through the vertical deformation and the shear of the coupling elements. As the tension wall (on the left in Figure 2.11b) axial load is reduced, so are its bending moment and base shear. To the contrary, in the compression wall (on the right in Figure 2.11b) the axial load is increased as well as its bending moment and base shear. An additional resisting moment mechanism appears between the two walls as they have different axial loads (which can be of opposite sign, as shown on Figure 2.11b, and are separated by a lever arm  $x_T$ . difference of axial load in the walls is of course directly linked to the shear capacity of the coupling elements and their stiffness.

The elastic behaviour of such system has already been studied in numerous studies, notably using a laminar analysis, and have proven the importance to consider those effects. The following results from a parametric analysis from [8], on a service core composed of two U-shaped walls, details all the implications coupling can have on the LLRS walls.

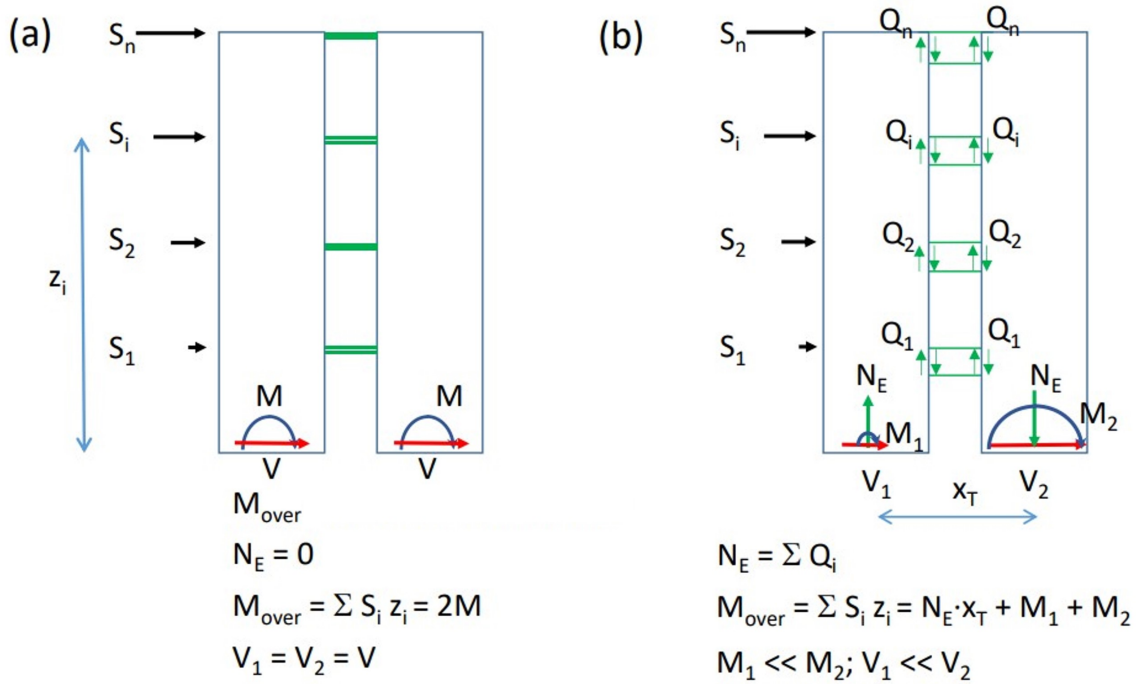


Figure 2.11: Coupled wall response. Source: [17]

From Figure 2.12(a) and (b) it is clear that the coupling has an influence on the resisting moment of both walls and on the added resisting moment through lever-arm action between the two walls. Stronger coupling induces, as expected, higher axial force  $T$  and therefore higher bending moment over-strength  $lT$ . Below a certain level of coupling, the system reverts to an uncoupled behaviour and above a certain level of beam depth (750 mm) and therefore of stiffness, the gain is not significant as we tend to an infinite stiffness assumption.

With the increase of axial load in the compression wall, the axial deformation is more important towards the top of the wall. A consequence of this is the significant drop of laminar shear at the top of the system in the case of a strong coupling, as seen in Figure 2.12. At the bottom, low laminar shear translates the fact that it is in fact the curvature of the walls which causes this shear, and it is thus tending to zero unless the coupling is considered perfect.

Figure 2.12 (e) displays the main benefit of walls coupling as being the better control of deformation in the elastic regime.

However, during an earthquake event plastic hinges will form at the base of the walls and therefore a redistribution of forces will occur, transferring the load from the tension to the compression wall. [8] suggest considering a redistributed bending moment between the walls of maximum 30%. Large amount of energy can also be dissipated thanks to the yielding of the coupling elements where a redistribution can occur as well, the same

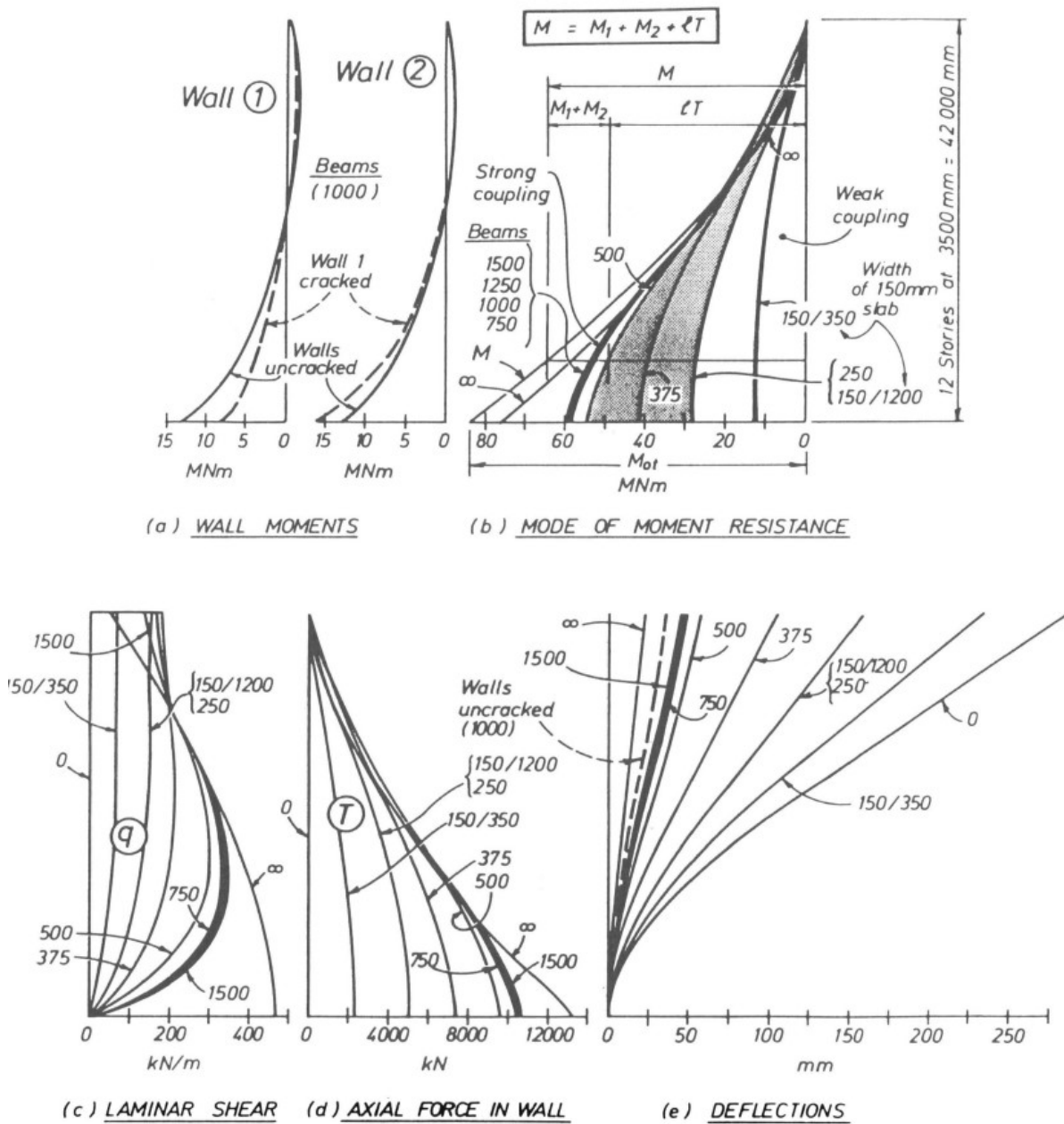


Figure 2.12: Structural response of coupled shear wall-slab structure. Source : [8]

authors suggest that it should not exceed 20% of the predicted shear by the elastic analysis. To achieve this, coupling elements need to be design and detailed for quite large plastic deformations.

The coupling mechanism is mostly defined for coupling beams, as they are the best way to interconnect two walls. The stiffness, resistance, and deformation capacity of those elements being a key to the effectiveness of a coupling system, slabs are regarded as a weak way of linking different walls. The Eurocode 8 even advises designers to not take into account the coupling through slabs :

*Coupling of walls by means of slabs shall not be taken into account, as it is not effective.*

However, a number of studies have shown that in some cases the coupling through a slab can in fact take place and that neglecting it could cause significant errors in predicting the behaviour of such a structure. The previously shown parametric analysis even considers different slab dimensions and there is a notable impact on the behaviour, even if it is less significant than for the larger beams.

In a large-scale shake table test including a wall coupled to a flange wall through a slotted slab [18], the influence of the coupling system was clearly noted. The over-strength in bending moment as well as the increase of shear in the wall were related to the added axial load from the shear of the slab and even was the source of some errors in the submissions to the blind prediction contest.

Another study [19] seems to show that the behaviour of a slab coupled walls system gradually transitions from cantilever to frame action as the slab stiffness increases. A more recent shake table test study [17] reports that up to 53% of the resisting moment was provided thanks to the frame action, placing it way above the 25% mark of EN1998-1:2004 and therefore classifying the system as coupled even though the walls are linked by a slab.

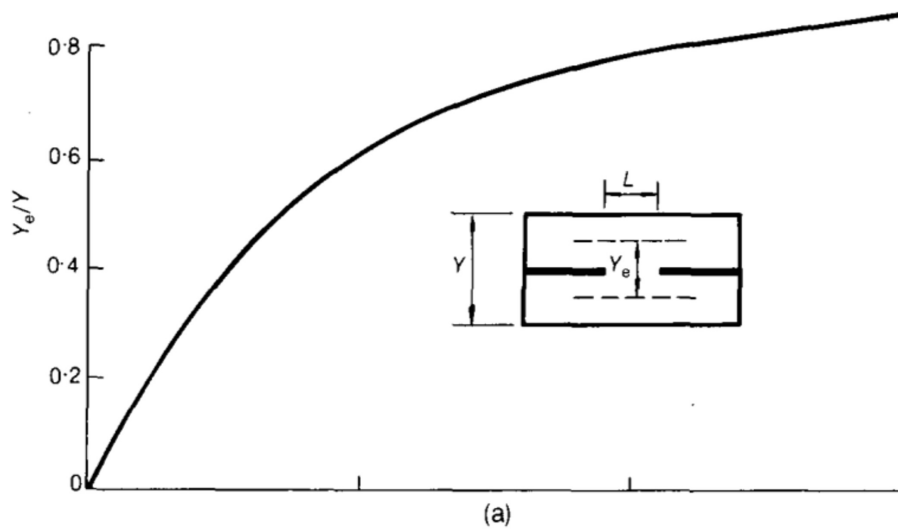


Figure 2.13: Effective width of the slab,  $Y$  is the spacing between two walls and  $Y_e$  the effective width. Source : [20]

#### 2.4.2.1 Effective stiffness of the slab

The assumptions used in the coupling beam systems would be definitely useful to compute the possible effect of the slab, provided that its actual out of plane bending and shear stiffness can be estimated. [20] proposes different effective width empirical formulas depending on the section shape of the walls, for example the effective width of a coupling slab between two rectangular walls is defined in Eq. 2.14 and is shown in Figure 2.13:

$$\frac{Y_e}{Y} = \frac{L}{Y} \left( 1 - 0.4 \frac{L}{Y} \right) \quad (2.14)$$

In contrary to coupling beams, slabs are subject to punching shear. This can prove critical, mainly if the slab is poorly detailed to resist such action. Slab coupling induces high stress at the junction of the wall and the floor diaphragm, therefore those regions could see the appearance of punching shear problems. However, this aspect will not be treated in this thesis.

In Chapter 6, a numerical analysis will be lead in order to verify some of those findings and to try to assess the relevance of taking the coupling through a slab into account for the design of reinforced concrete shear walls.

### 2.4.3 Stirrup effect

This interaction is actually the core of this thesis and the main subject behind its quite general title. It is also the less covered interaction between walls and diaphragm in the existing literature. The principle behind this effect illustrated in Figure 2.14, as its name implies, related to the fact that the floor diaphragm could act like a discrete stirrup for the wall and prevent diagonal cracks from opening at this location by restraining the horizontal deformations.

The diagonal cracks developing in a reinforced concrete shear wall subject to horizontal loading cause a deformation of the section in the form of an elongation of the depth to allow those cracks to open. Such flexural-shear behaviour and cracks are very common in the real behaviour of walls, as seen in Section 2.1.1. Design and verification of shear wall are generally done by isolating the wall from its surrounding structural elements and checking the different limit states, as well as assessing its failure mode separately.

However, as displayed in Figure 2.14, if the transfer of inertial forces can be computed based on the mass of the structure as explained in Section 2.2, the restraining forces at play in this case do not appear directly in the static model of forces applying on the wall. Considering this restraint, if the diagonal cracks were to reach the first floor diaphragm, the latter would act as a sort of very large and very stiff stirrup, preventing the cracks from opening at this section. At the direct proximity of this diaphragm, it would also be more difficult for the cracks to open and widen. Cracks could also follow a different pattern and have different orientation.

If the stirrup effect is significant, it could even prevent a pre-emptive shear failure to occur due to horizontal tension of the stirrups. On the other end, this restraint could increase the compression of the diagonal concrete strut and cause an earlier shear failure in compression. A flexure-shear failure mode could then be transformed into a pure shear or pure flexural failure, as the case may be.

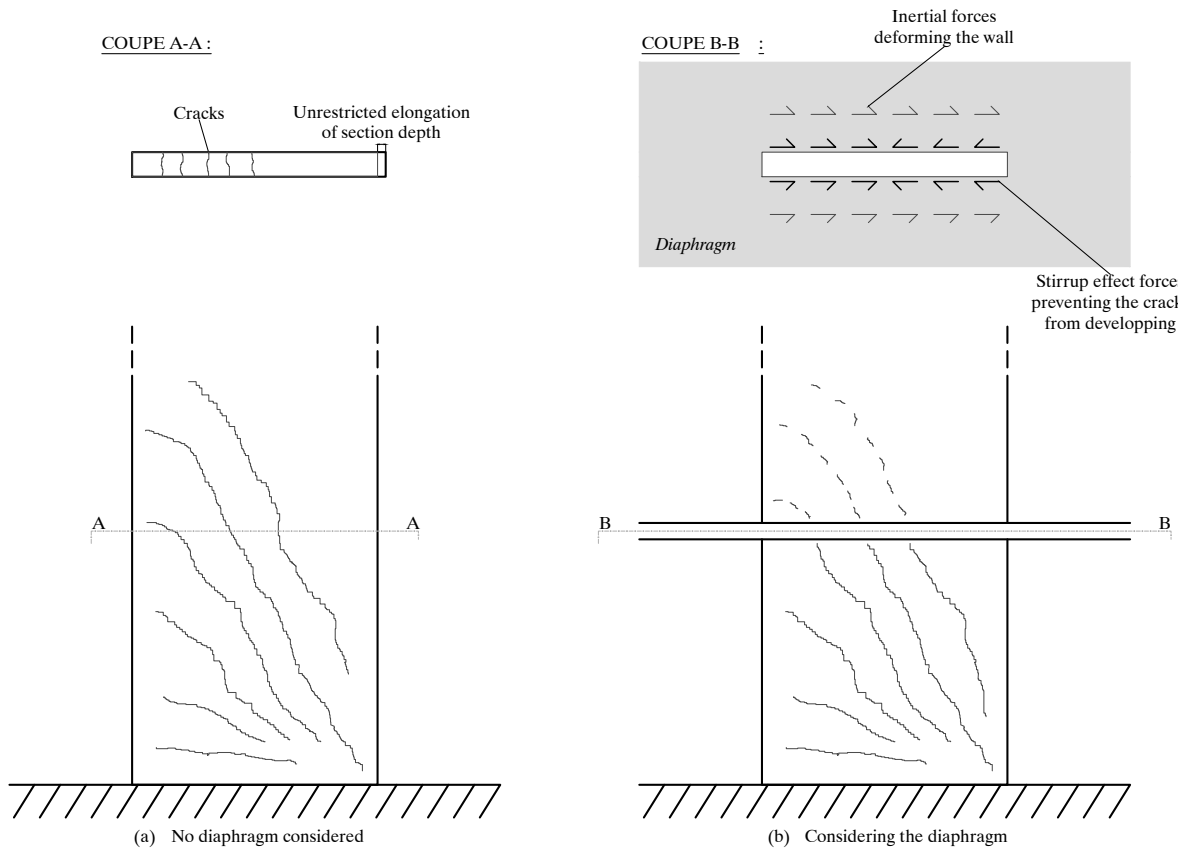


Figure 2.14: Illustration of the stirrup effect.

This interaction is intrinsically linked to the shear behaviour of shear walls. It also depends highly on the fact that the floor diaphragms needs to be sufficiently low to have an actual influence on the cracks and on the behaviour of the wall.

Investigating this phenomenon is precisely the goal of this thesis, and a numerical analysis is presented in Chapter 5

## Chapter 3

# Numerical model

### 3.1 General modelling approach

As the goal of the numerical simulations will be to observe the load-displacement relationship of the shear walls and to determine the failure mode and configuration, different modelling approaches could be chosen, most of them have been discussed in [21].

The model needs to be able to depict the global non-linear behaviour of the wall, but also the more local properties linked to the influence of the diaphragm, such as the stirrup effect mentioned in the previous chapter. The shear behaviour of the wall is very important in this study and needs to be correctly captured by the chosen model. For this reason, standard plastic hinge analysis as described in the Eurocode 8 and in [22] is inadequate.

Models based on a truss analogy have been developed by discretizing the wall using uni-dimensional struts and ties, as in [23]. Wisely defining the geometry of such models is not an easy task. As it uses the diagonal strut to represent the compressed concrete, its effectiveness to predict shear failure due to tension in the web is limited and not straightforward. For these reasons, this model was not suitable for this work.

Flexure-shear interaction are at the centre of certain multi-spring approaches such as shear-panel-element and multiple-vertical-line models presented in [24] and [25].

Kinematic models have also been elaborated, such as the three-parameters kinematic model proposed in [26] by Mihaylov et al. While it can capture quite well the behaviour of shear-dominated walls, its use in this thesis does not seem appropriate.

Finally, given the certain difficulty to accurately capture the shear-flexure interaction in the behaviour of shear walls and the relatively complex nature of the stirrup effect to be observed, the finite element (FE) approach seem to be the most relevant and the best fitting for this work. This approach allows the model to consider a wide range of degradation and failure mechanism [27]. In accordance with the experience of the

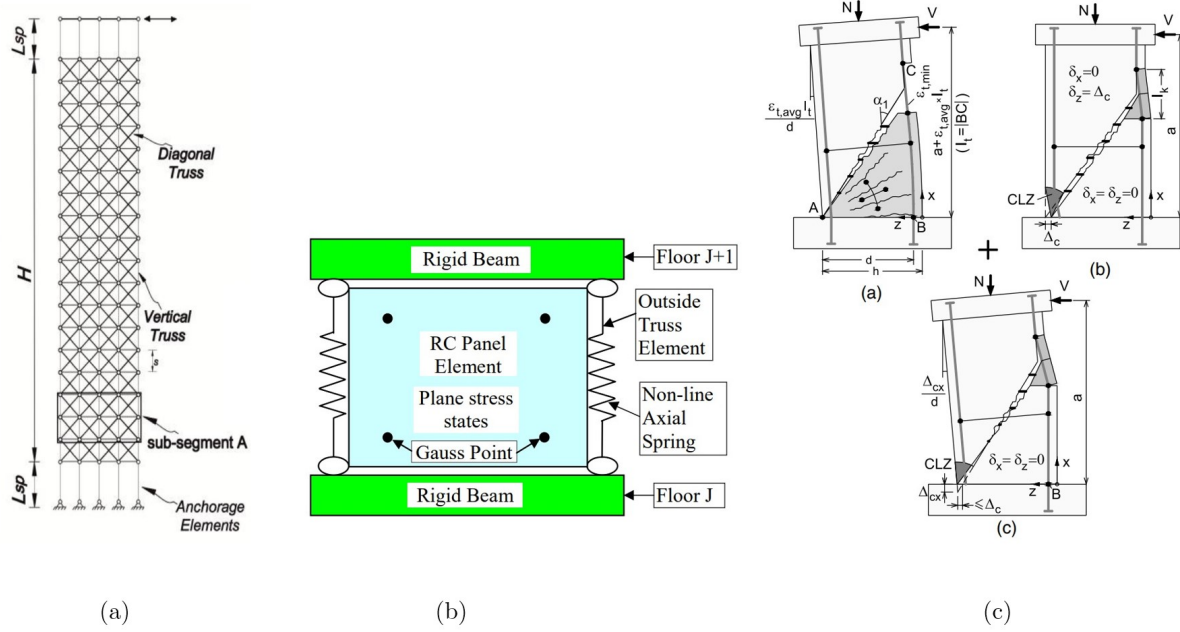


Figure 3.1: (a) Truss analogy - source: [23], (b) RC shear panel and truss model - source: [24], (c) Three-parameter kinematic theory model - source: [26]

promoters of this thesis, the FE software Vector2 has been chosen to pursue the numerical analysis.

## 3.2 Finite elements and solution procedure

Vector2 is a nonlinear finite element analysis tool for reinforced concrete planar elements, it has been elaborated by a team led by Pr. Frank J. Vecchio from the University of Toronto. It treats many complex aspects of the behaviour of reinforced concrete, and therefore is perfectly suited to capture both the stirrup effect and the coupling mechanism of shear walls.

This software uses displacement based, plane stress elements to model the concrete. The bases of the Vector2 analysis - Modified Compression Field Theory (MCFT) and Disturbed Stress Field Model (DSFM) - have been discussed in numerous papers, notably from Vecchio in [28] and [29]. The MCFT theories aim at predicting the load-deformation response of the reinforced concrete finite elements. The concrete is considered as an orthotropic material and smeared with cracks which can rotate depending on the stress and strains the element is subjected to. This theory relies on three different relationships, illustrated in Figure 3.2 :

1. Compatibility
2. Equilibrium

### 3. Constitutive

The DSFM completes the MCFT by addressing some of its flaws. It improves the computation of crack shear slip deformations by calculating them explicitly and therefore improves the computation of shear stiffness and strength when compared to the MCFT.

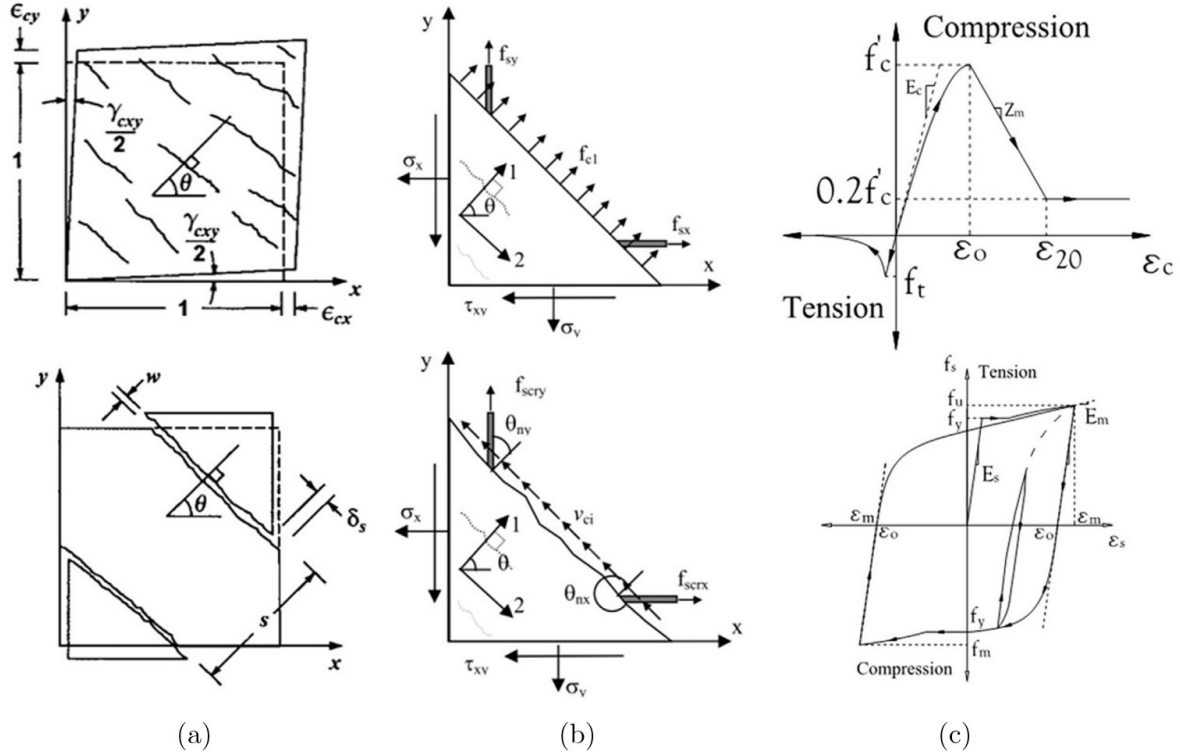


Figure 3.2: Three relationship at the base of the MCFT-DSFM theory. (a) Compatibility: net strain (above), slip strains (below) ; (b) Equilibrium: between cracks (above), at crack location (below) ; (c) Constitutive: concrete (above), steel (below). Source : [27]

The full finite element procedure is explicitly detailed in [30] and the main algorithm is shown in A.2. It follows a very standard iterative process for displacement based elements. The convergence criterion in this numerical analysis has been however set to displacements - weighted average, differences with the criteria presented in the paper (secant moduli) are negligible.

## 3.3 Material models

### 3.3.1 Concrete

Vector2 offers a very wide range of material models subtleties, especially for concrete, allowing the user to fine-tune its approach of the concrete modelling to better fit the phenomenon he or she wishes to observe. Table 3.1 summarizes the most relevant concrete models used for this numerical analysis.

| <i>Model:</i>          | <i>Model selected</i>          |
|------------------------|--------------------------------|
| Compression Pre-Peak:  | <b>Popovics (NSC)</b>          |
| Compression Post-Peak: | <b>Base Curve</b>              |
| Compression Softening: | Vecchio 1992-A (e1/e2-Form)    |
| Tension Stiffening:    | <b>No Tension Stiffening*</b>  |
| Tension Softening:     | Nonlinear (Hordijk)            |
| Confined Strength:     | Kupfer / Richart               |
| Dilatation:            | Variable - Isotropic           |
| Cracking Criterion:    | Mohr-Coulomb (Stress)          |
| Crack Stress Calc:     | Basic (DSFM/MCFT)              |
| Crack Width Check:     | <b>Agg/2.5 Max Crack Width</b> |
| Crack Slip Calc:       | Walraven                       |

Table 3.1: Constitutive models used for the concrete material. Models different from the default option in bold.

Most of the selected models are the ones proposed by default in Vector2 as they are supposed to capture pretty well the widest range of concrete behaviours.

After discussions with the promoters of this thesis and based on experience from previous researches like [31], the *Compression Pre-Peak* and *Compression Post-Peak* models used are the ones proposed by Popovics in [32].

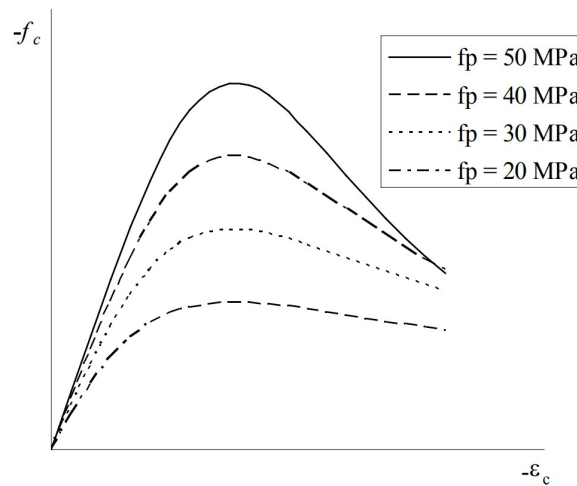


Figure 3.3: Popovics constitutive compression law for concrete.  
Source : [30]

By default, the tension stiffening model is proposed as the Modified Bentz model (2003), described again in [30]. However, for reasons developed in Section 4.5.1 about the localization phenomenon, it was not used in the different models. Tension stiffening is therefore

not considered. It does not have a negative impact on the reliability of the results, since this model is most suited when discrete reinforcements are used. Tension stiffening is actually referring to the fact that after cracking, the concrete stress virtually goes to zero at the cracks location. However, average tensile stress still remain in the concrete between the cracks due to bond action with the reinforcements. The stiffness of the reinforcement bonded to this concrete is then greater than the one of the reinforcement alone. As mentioned, no discrete reinforcements were used in the models and therefore the bond action is not taken into account and neither is the tension stiffening. Not considering it actually did not greatly modify the force-displacement curves but prevented strain localization to systematically occur right after the inelastic domain of the force-displacement curve is entered.

### 3.3.2 Steel reinforcements

To ease the modelling of the numerous models constructed in the scope of this thesis, all steel rebars have been modeled as smeared reinforcements in the concrete. The concrete finite elements described in the previous section are then composed of a hybrid material which integrates both the concrete and the smeared reinforcement being uniformly distributed in the different elements. Dowel action, buckling of the reinforcements and bond effects are therefore not considered and their models are not used in the simulations.

Steel is therefore not used as a main material in the different walls but will be used in some models as an additional structure to apply a certain load case.

## 3.4 Load application method

Actions due to earthquake event are not monotonic and cyclic analysis is often used to capture seismic behaviour of structures. However, to simplify the modelling, only monotonic action will be studied here. It fails to capture some effects of earthquake such as fatigue, but it remains a good way to assess the displacement capacities of RC walls. The curve of such monotonic loading analysis can easily be correlated to an envelope of values from a cyclical one as will be done for the validation process in Chapter 4

Two main options are offered to the user in order to apply a load case on a numerical specimen. External forces or imposed displacements can directly be applied at multiple point of the structure.

When applying external forces, the computation is much faster and has a better convergence since the elements used are displacement based but it of course does not allow capturing the post-peak behaviour of the structure regarding its force-displacement evolution. Once the maximal load is reached, the next steps can not be relied on, as the numerical model generally produces unrealistic results. If a wall has sufficient ductility and does not fail before reaching its peak resistance, the applied force method can not safely determine its failure mode nor its ductility. It is therefore required to approach

the maximal loading point with caution and a small load step to ensure there are enough data points before the results become untrustworthy.

On the contrary, post-peak analysis of the behaviour of a structure is entirely possible if the loading applied consists of an imposed displacement. Imposing a displacement is also at the heart of the pushover analysis mentioned in Section 2.2 it is a very good way to predict the ductility and the failure mode of a wall. For all those reasons, the imposed displacement loading will be used in this numerical analysis and applied as a monotonically increasing function. The load step is selected to be sufficiently precise, but also to limit the computation time to relatively acceptable values (from 1mm per step to 3mm per step in some cases).

It is worth noting that comparing the two loading methods on a same shear wall model can confirm the different constitutive models and boundary conditions of this model, such verifications were then made in the validation phase of the software.

## 3.5 Dependability

Even if the software is very complete and aims at capturing the precise behaviour of reinforced concrete thanks to a wide range of different material models and other parameters, the trustworthiness of the finite elements methods is still variable. Different studies [31] and [33] have pointed out that such models can sometimes, even if calibration seem to have been correctly implemented, give unrealistic results and capture the right behaviour only up to a certain point. Strain localization, mesh sensitivity and convergence are all issues impacting the dependability of the results as discussed in the previously mentioned studies and in Section 4.5.

It is therefore important to define the different criteria which can detect the failure during the FE analysis. They will help assess the failure mode of the different wall in the numerical simulations. Different types of criteria can either define a certain configuration as a physical failure or assess the next load steps as unreliable due to numerical issues.

### Global behaviour

A commonly used criterion to assess the failure of a shear wall is simply whether it can sustain sufficient loading compared to its peak resistance. If a wall resistance drops below 80% of its maximal resistance, it can be considered that the wall has failed.

#### 3.5.0.1 Local behaviour, material related

The different material models of course possess ultimate strains and strengths. However, the numerical simulation does not stop when those material limits are reached. It is up to the user to pay close attention to the local behaviour of the model, even if its general

behaviour seems plausible, and to doubt any results which would exhibit unrealistic local strain, for example. To this end, different criteria are defined to detect material failure.

### Reinforced concrete

Crushing concrete is detected when the concrete reaches its ultimate strain. Standard concrete ultimate strain of  $\epsilon_{cu}$  is usually taken as 0.35% as seen in Figure A.1 from [34]. However, in most walls, concrete is confined using reinforcement hoops to enhance their compression resisting properties. Confining concrete increases both its maximal resistance  $f_{ck,c}$  and its ultimate strain  $\epsilon_{cu,c}$ . EN1992-1 proposes the following value for  $\epsilon_{cu2,c}$ :

$$\epsilon_{cu2,c} = \epsilon_{c2,c} + 0.2 \frac{\sigma_2}{f_{ck}} \quad (3.1)$$

Where :

$\epsilon_{c2,c}$  is the yield strain of confined concrete and is equal to  $\epsilon_{c2}(f_{ck,c}/f_{ck})^2$

$\sigma_2$  is the effective lateral compression at the ULS caused by confinement reinforcements

EN1998-2: 2005 proposes the following formula for  $\epsilon_{cu,c}$ :

$$\epsilon_{cu,c} = 0.004 + 1.4 \frac{\rho_s f_{ym} \epsilon_{su}}{f_{cm,c}} \quad (3.2)$$

With:

$\rho_s$  is the volumetric ratio of confinement reinforcement

$\epsilon_{su}$  is the mean value of the reinforcement steel elongation at maximum force  $\epsilon_{um}$

The models in Chapter 5 did not include explicitly boundary zone confining reinforcement hoops. However, using the unconfined value of  $\epsilon_{cu} = 0.35\%$  seemed too conservative, as there still are transversal reinforcements encasing concrete on three sides and providing some confinement. The formula was then used with the transverse reinforcement ratio as  $\rho_s$ . For example, a transverse reinforcement ratio of  $\rho_t = 0.185\%$  will translate to an ultimate concrete strain of 5.3%.

### Steel reinforcements

Ultimate tensile strain  $\epsilon_{su}$  is a property of the chosen steel reinforcements and therefore depends on the rebars chosen and is usually above 5% with certain reinforcements being capable of having ultimate strain of more than 10%. To account for fatigue which is not

considered in the monotonic loading models here, limiting the steel strain limit to 60% of  $\epsilon_{su}$  is preferable as suggested in[33].

## **Numerical criteria**

With all numerical simulations comes a certain uncertainty of results. Some numerical phenomenons can be completely independent of real physical behaviours, and they need to be detected. The main issues encountered in the numerical simulations of this thesis were : oscillations, convergence and strain localization.

### **Oscillations**

If results start oscillating without plausible physical cause, the results are to be discarded as the iterative algorithm can not converge. A solution could be to reduce the load step to better approach a convergence. It is not really a definitive criterion discarding all the following results, but rather an indication of distrust and that the results must be carefully analysed.

### **Strain localization**

This phenomenon is principally related to the tension behaviour of the element in this particular case. While strain localization can physically occur and is a part of solid mechanics, numerical strain localization can produce results entirely unreliable. If an unjustified strain localization is noticed, the next calculation steps are to be considered accordingly. This topic is more precisely detailed in Section 4.5.1.

## Chapter 4

# Validation

To ensure the reliability of the models selected for this analysis, it is imperative to compare numerical results to real life results observed in different test studies. To that end, different shear wall test researches have been selected to validate the numerical parameters of the finite elements software.

With the shear behaviour and the diaphragm effect in mind, it was important to select walls who would be concerned by those two aspects. It was also decided with the promoters of this thesis to focus on walls with the highest aspect ratio possible.

Walls CW0, CW1 and CW2 from [31], walls VK1 and VK3 from [12] as well as all SW3 from [35] were selected to conduct this validation.

### 4.1 Tatar - 2020

Research on the shear behaviour of reinforced concrete walls was conducted at Université de Liège by N. Tatar in 2020 under the supervision of B. Mihaylov [31]. Different walls were tested in laboratory and analysed using both the three-parameter kinetic theory mentioned in 3.1 and finite elements models in Vector2. This is therefore a good way to compare the established model with both laboratory testing and existing FE analysis results of those same walls.

Those 2.3 m high walls presented the same transverse reinforcement ratio  $\rho_t$  of 0.15%. Walls CW1 and CW2 had vertical reinforcement ratio  $\rho_l$  of 0.72 where CW0 was more reinforced at 0.79 %. Axial load ratios were of 0%, 10% and 21% of concrete section capacity  $bhf_c$  for CW0, CW1 and CW2 respectively.

Accordingly to the test procedure, imposed displacement was applying on the model through a stiff steel plate to dissipate the actual imposed boundary condition. As it can be seen on Figure 4.1b, the foundation block is not taken into account to simplify the model. The strain penetration is therefore not considered, as the wall is fully clamped at its base. The axial load is represented by a distributed uniform load at the top of the

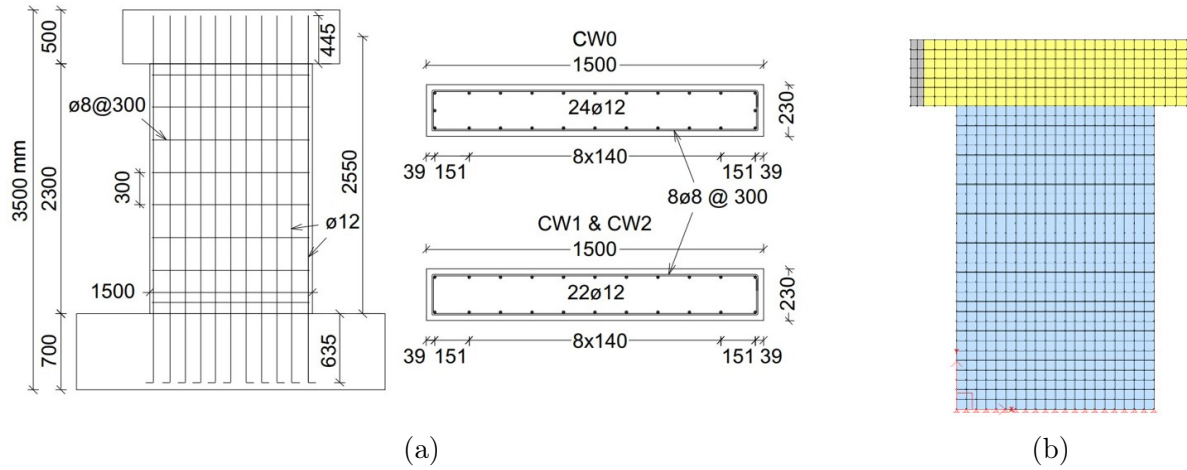


Figure 4.1: (a) Test specimens CW0, CW1 and CW2 dimensions and reinforcements. Source : [31], (b) Finite element model in Formworks, the pre-processing software of Vector2.

specimen. Elements size has been defined such as there are 20 elements along the length of the wall as suggested by N. Tatar in his study, however a mesh sensitivity analysis will be detailed in Section 4.2.1

## Force-displacement curves

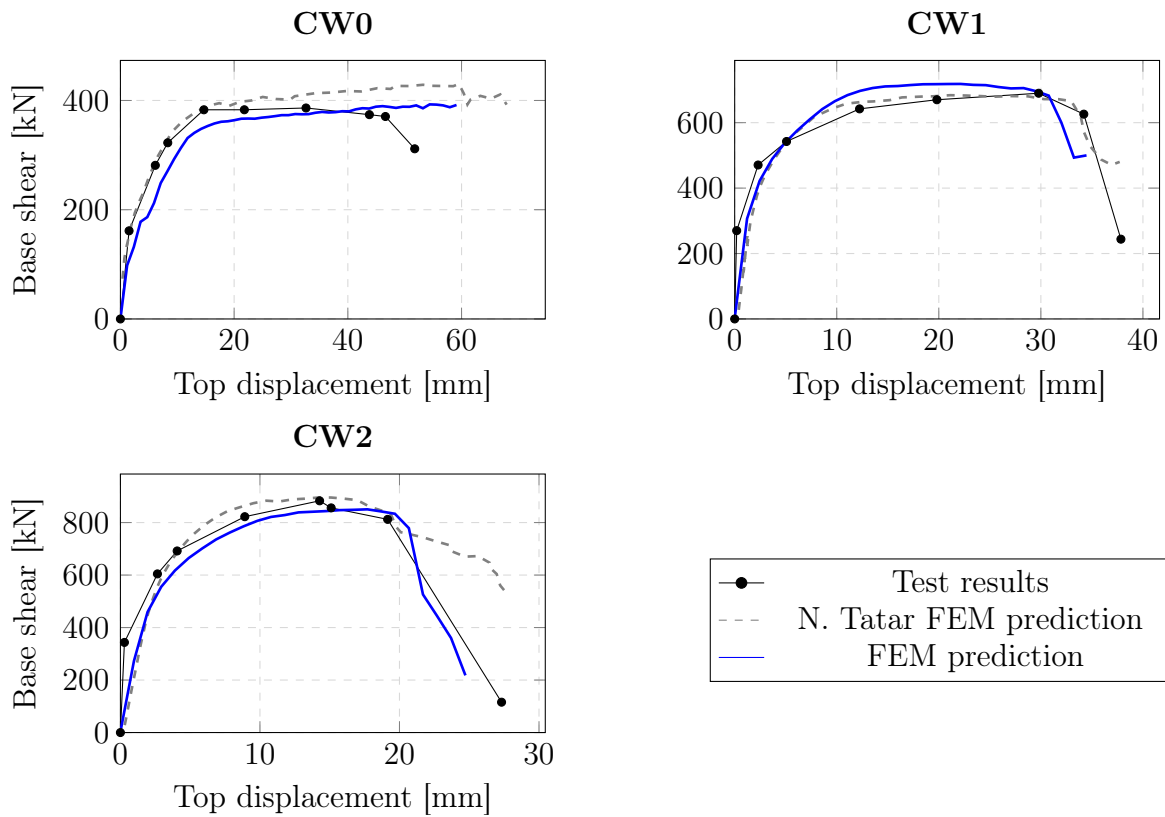


Figure 4.2: Comparison of the force-displacement curves of the laboratory test, FEM results from [31] and FEM predictions

The force-displacement curves from the numerical model seem to very well fit the laboratory test results for walls CW1 and CW2, as shown in Figure 4.2. For CW0 however, the model shows a larger displacement capacity. Lateral resistance is also well predicted by the different FEM with respectively +2%, +4% and -4% differences for the three walls.

#### 4.1.1 Failure modes and cracking pattern

The predicted cracking pattern at the peak resistance is really consistent with the observed results of the test, as shown in Figure 4.3. Orientation of the cracks and the positioning of the main ones is quite well predicted by the model.

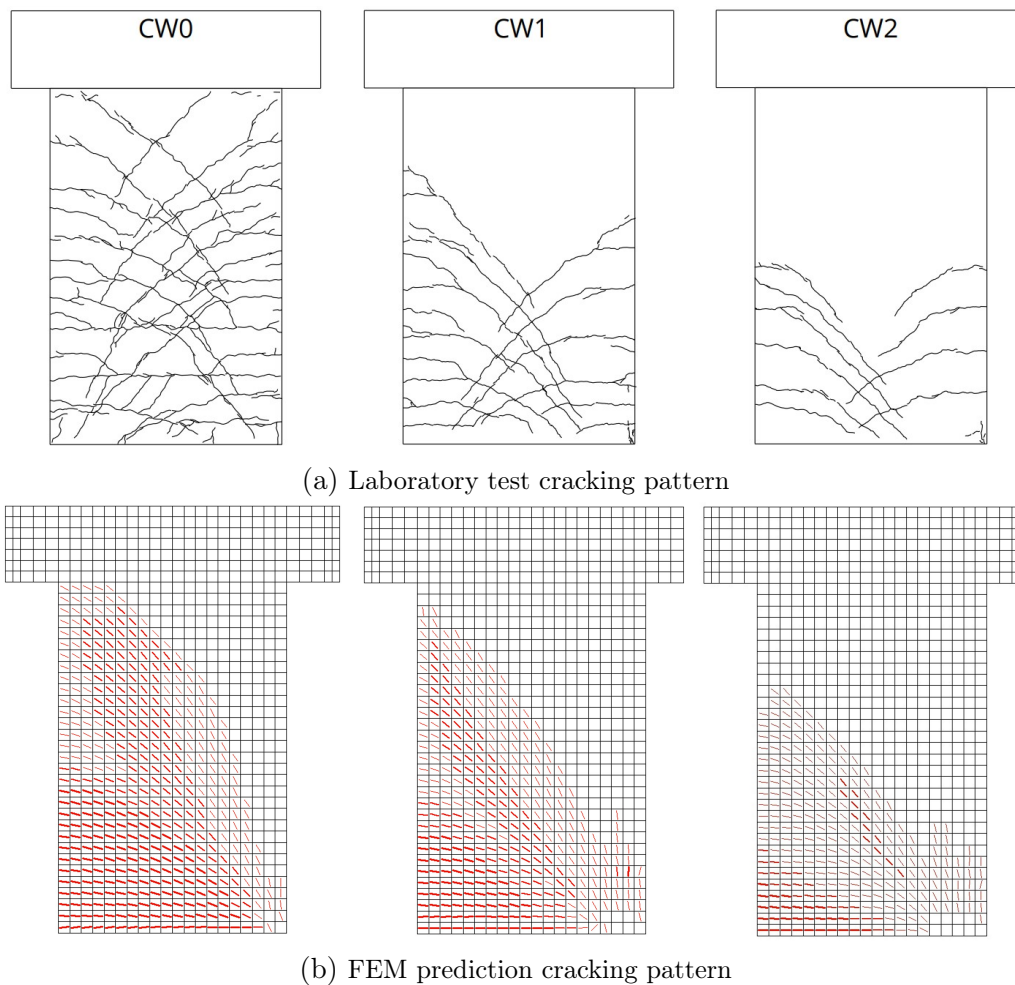


Figure 4.3: Crack pattern at peak resistance comparison between laboratory testing and FEM prediction. (a) Source : [31]

Wall CW0 failed due to rupture of the longitudinal rebars in tension during the testing. Buckling of the compression reinforcements was also observed. The FE model however did not capture the rupture in tension of the rebars but only the compressive failure of the concrete at the bottom right. Both walls CW1 and CW2 failed due to the same crushing of concrete at the compressed toe and buckling of reinforcement was also observed, being more important for wall CW2 since it was submitted to higher axial load. Those failure

modes also correspond to the ones observed by the conducted FE analysis and confirm the ability of the model to correctly predict this type of failure mechanism.

## 4.2 Bimschas - 2010

Three different bridge piers were tested as part of a study conducted at ETH Zurich by M. Bimschas in 2010 [12]. The investigation was about bridge pier seismic behaviour, but the specimens used could entirely be considered as being shear wall part of a building, and therefore they were considered relevant in this study.

The interest in this study resides mainly in the fact that even for aspect ratios of around 2.2, walls VK1 and VK3 experienced a combined shear and axial load failure. It is in part due to the low amount of transversal reinforcement in the web.

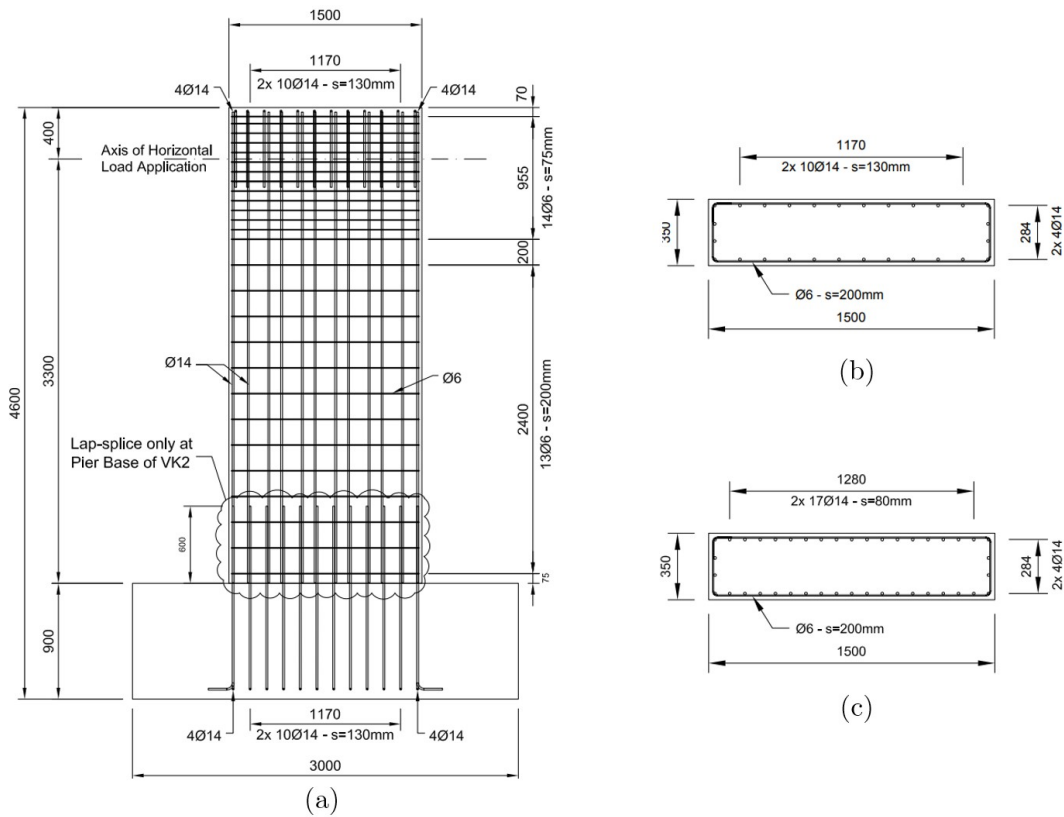


Figure 4.4: (a) Dimensions of specimens and reinforcement of specimens VK1 (b) and VK3 (c). Source : [12]

The specimens were half-scale bridge piers of 3.7 m high ( $L_v = 3.3$  m) with low transverse reinforcement ratios of 0.08% and longitudinal reinforcement ratios of 0.82% for VK1 and 1.23% for VK3. Low ALR was applied on the walls at 6.7% and 7.7% for VK1 and VK3 respectively.

### 4.2.1 Mesh sensitivity

The size and shape of the finite elements can entirely be decided by the user in FormWorks. This of course plays a role in the computation demand of the software and the machine running it. It can also influence the results quite significantly. Figure 4.5 shows the difference in force-displacement response when changing the size of the elements.

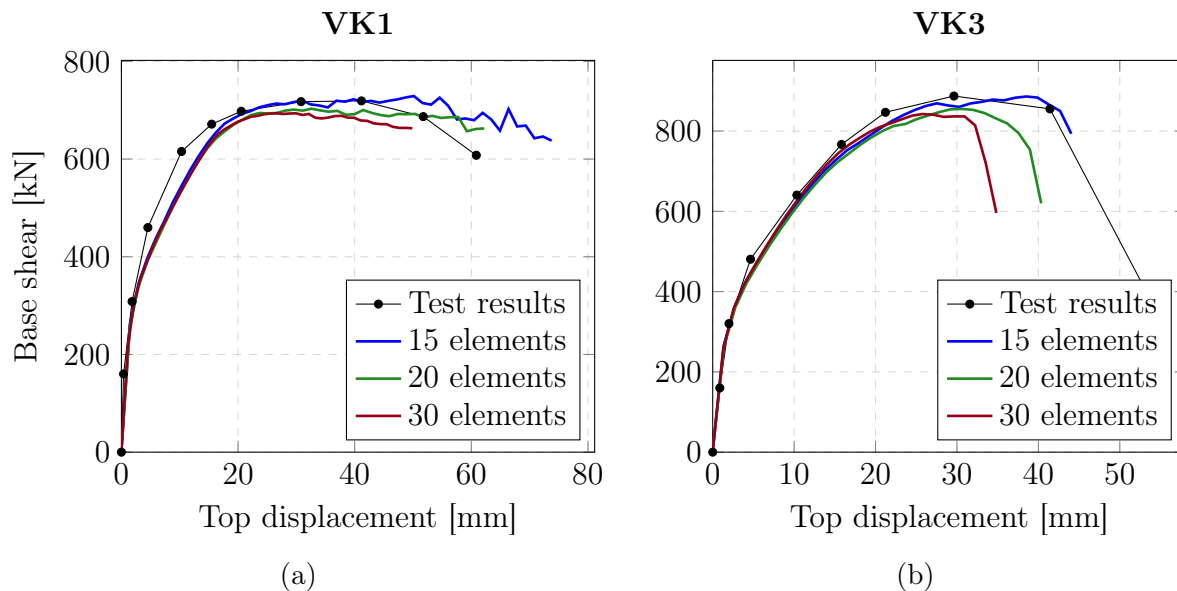


Figure 4.5: Mesh sensitivity analysis of the force-displacement response for VK1 (a) and VK3 (b)

The horizontal load resistance of both walls is very well predicted by the FE model with from +1% to -4% for VK1 estimations depending on the element sizes and from -0.1% to -5% for VK3.

Intuitively, refining the FE mesh is expected to bring precision to the results, as it is often the case in such analysis. However, we can see on Figure 4.5b specially that using smaller elements drastically reduces the displacement capacity of the model. For VK3 for example, going from 100x100 mm elements (15 elements along the length of the wall) to 75x75 mm elements (20 elements along the length) reduces the displacement capacity from 44 mm to 40.3 mm, a drop of around 9%. Using an even finer mesh of 50x50 mm elements further reduces the ductility to a maximal displacement of 34.8 mm, another drop of around 14%. The VK1 model shows similar effect of the mesh refinement. In the case of VK3, the 15 elements model seem to be the better approximation of the displacement capacity, whereas for VK1 it is the 20 elements model. This reduction was also observed in [31] and is caused by a phenomenon called strain localization, which is explained in Section 4.5.1. This numerical behaviour actually reflects a real physical effect, but in some cases it can produce appreciation errors and even cause a model to be untrustworthy past a certain point.

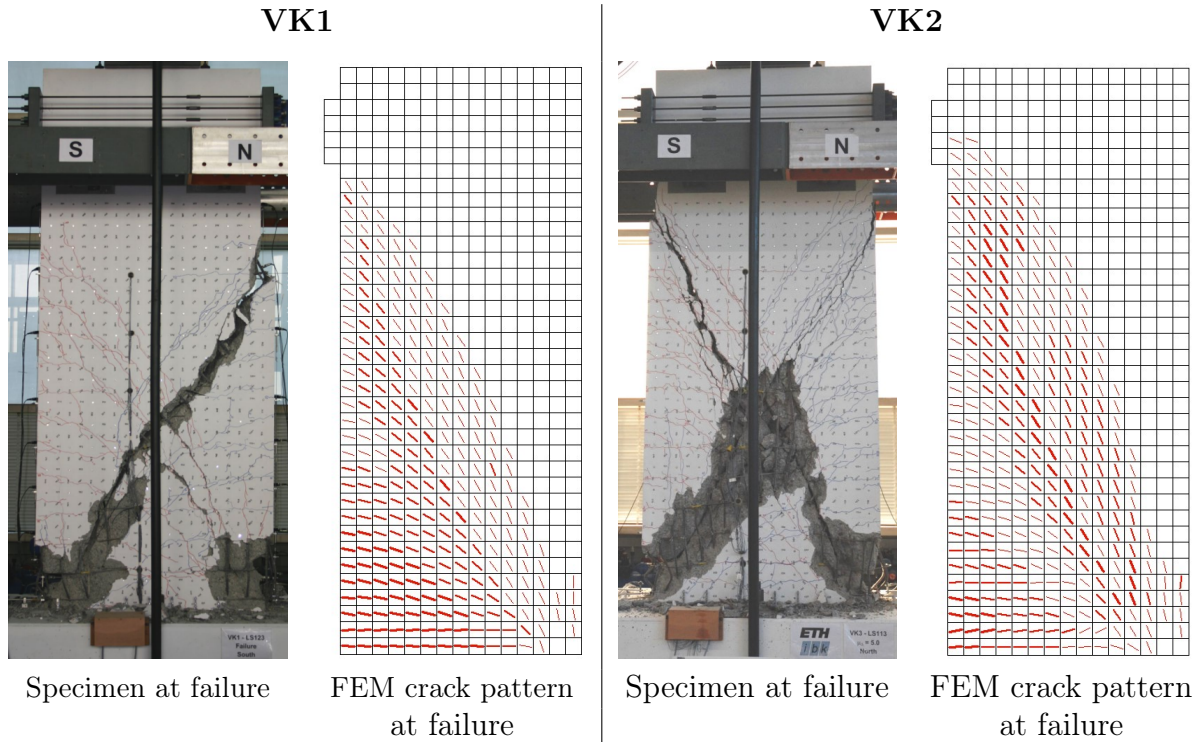


Figure 4.6: Specimen and FEM at failure

### 4.2.2 Crack pattern and failure mode

The predicted crack pattern is quite well predicted by the FE model as shown on Figure 4.6. Orientation of the cracks and the positioning of the main ones is quite well predicted by the model.

Wall VK1 and VK3 both failed due to a combined shear-flexure mechanism. VK3 presented a smaller ductility because of its increased longitudinal reinforcement and the shear cracks had already opened way before the ultimate failure, at which the main shear cracks were at some places 5 mm wide. On another end, VK1 seemed to follow a flexure dominated behaviour up until the very last loading steps where a large diagonal shear crack appeared and precipitated the failure of the specimen.

Aside from the purely numerical validation, observations were made during the laboratory tests reflecting a widening of the sections of walls VK1 and VK3. Considering that the stirrup effect would restraint this widening, it makes sense to imagine that a floor diaphragm placed somewhere along the diagonal crack would have prevented or slowed the shear failure.

The FE model of VK1 failed to really capture the abrupt apparition of the shear crack as it is rather constantly developing, but captured correctly the crushing of the concrete toe. For VK3, the large diagonal crack was more pronounced accordingly to the laboratory testing and the shear critical behaviour was correctly captured.

### 4.3 Du et al - 2020

A study conducted in 2020 by K. Du et al in Harbin, China was noticed for the fact that it had built a specimen consisting of the two lower floors of a RC shear wall [35]. Two horizontal actuators were used to impose displacements at the location of these presumed floors and the floor slabs were even partly cast. As the stirrup effect is expected to play a role mainly at the first floor, this configuration is rather close to the one which will be used to analyse the said phenomenon and the tests have been modelled in finite elements in order to validate a possible dual dependent displacement load application method.

The tested walls were half-scale 3.6 m high specimens with length of 1.8 m and thickness of 100 mm. They all presented the same overall reinforcement ratios: boundary longitudinal reinforcements  $\rho_b$  of 1.7%, web transverse and longitudinal reinforcements ratios  $\rho_t$  of 0.55%. Axial load of the considered specimen SW3 was 18% and its aspect ratio  $M/Vh$  was 1.85.

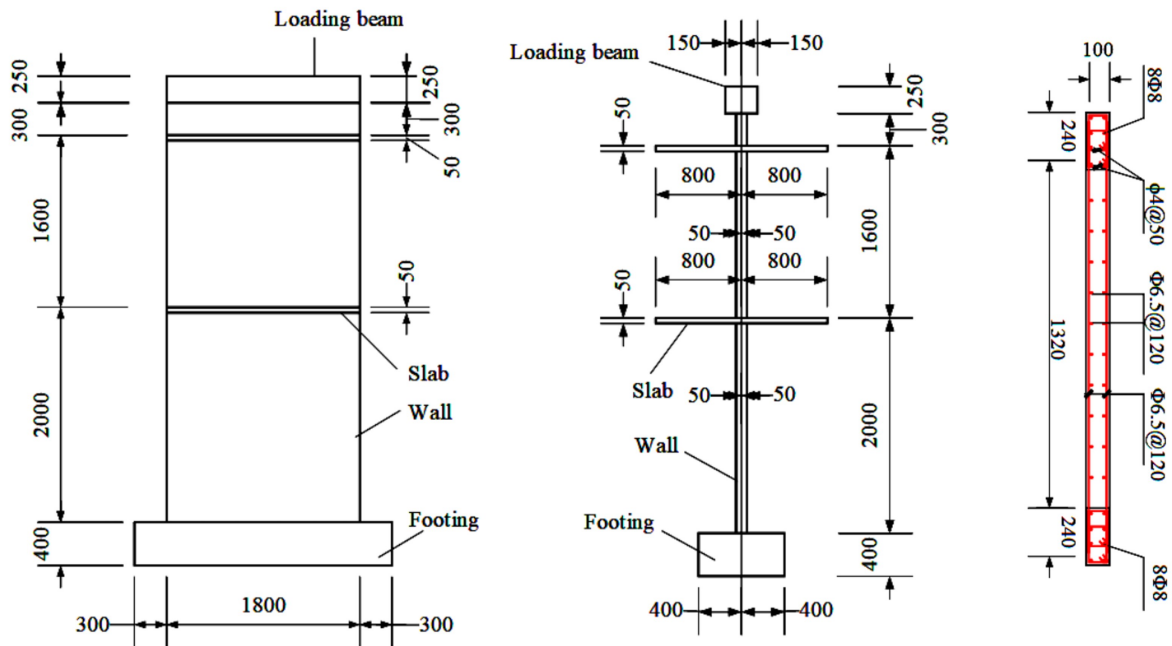


Figure 4.7: Test specimen dimensions and reinforcements of the wall SW3

The different colours on the model of Figure 4.8 depict the zones with different reinforcements and the different thickness of the floor slabs.

#### 4.3.1 Imposing two dependent displacements

The two horizontal actuators were actually operated through a force-displacement mixed control program, ensuring that the forces applied by the actuators would remain proportional (83.5% of the base shear  $V_b$  for the top storey and the remaining 16.5% of  $V_b$  for the bottom one. This actually proved to be a major difficulty in the finite element modelling as no relationship between two imposed displacements can be taken as input by Vector2.

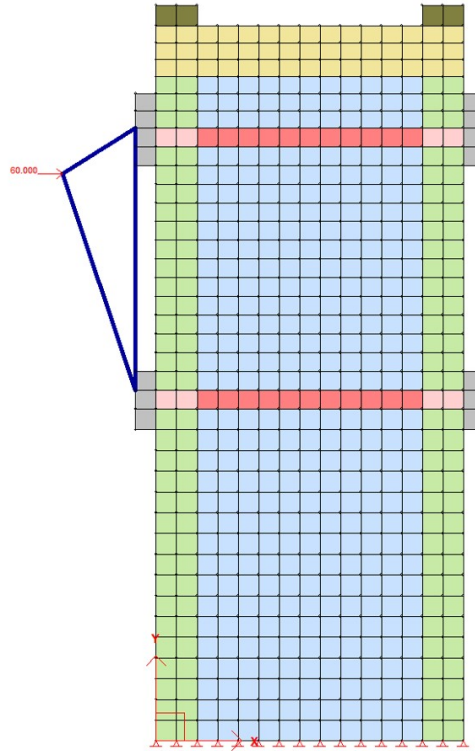


Figure 4.8: Finite element model of the SW3 test specimen with its additional structure

An additional structure would therefore need to be used in order to split one directly imposed displacement into two separate points while keeping the horizontal forces applied on the wall proportional. The blue triangle in Figure 4.8 is exactly that. When a displacement is applied on its left end, the two plates connected to the wall transfer this displacement to the wall as two separate displacements. The three nodes are pinned but as this truss structure is modelled using discrete reinforcements of thick  $8000 \text{ mm}^2$  section, it acts as an rigid structure.

Vector2 does not allow defining specific master-slave relationship between element and therefore, the truss remains fully pinned at both contact points to the wall. This results in an additional restraint preventing the left boundary region of the second floor to freely deform and elongate.

Such methods are not what Vector2 was conceived to analyse in the first place, and therefore this kind of models are of relatively low efficiency. They demand a way higher computation power than the simple displacement model presented in the previous section. For this reason, the computer used for the numerical simulations was not able to run models using both the additional structure and numerous elements. The mesh elements size was defined such as the length of the wall would be composed of 15 elements. This actually is quite fine considering the pretty good precision this amount of elements can achieve.

Comparison between a force application method and this displacement method was made to ensure that the influence of the truss was not skew the results. As seen in Figure 4.9, the global behaviour is the same for the force and the displacement loading up to the peak resistance of the wall model. This confirms the relatively low impact of the truss for the pre-peak domain. However, its main impact should be seen in the post-peak domain, where deformations are the largest, but comparing it to a force imposed method is not possible as it can not capture the post-peak behaviour.

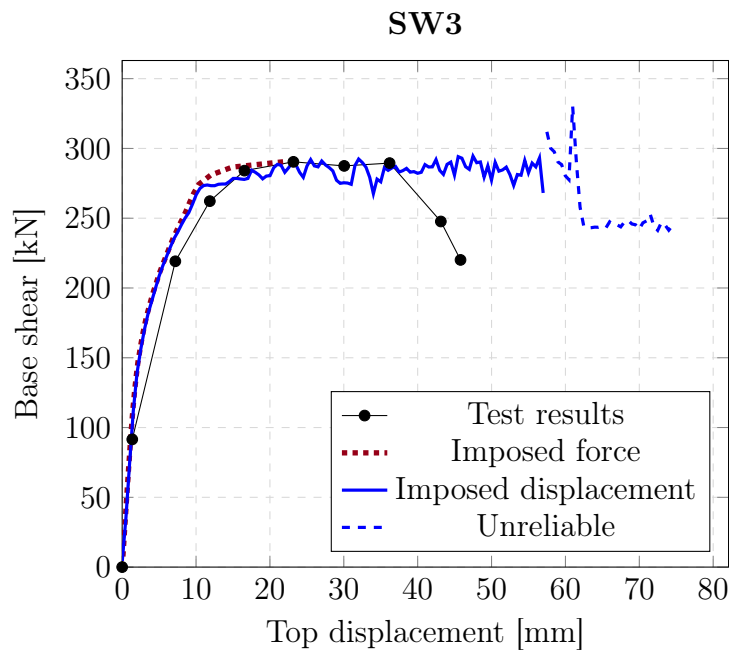


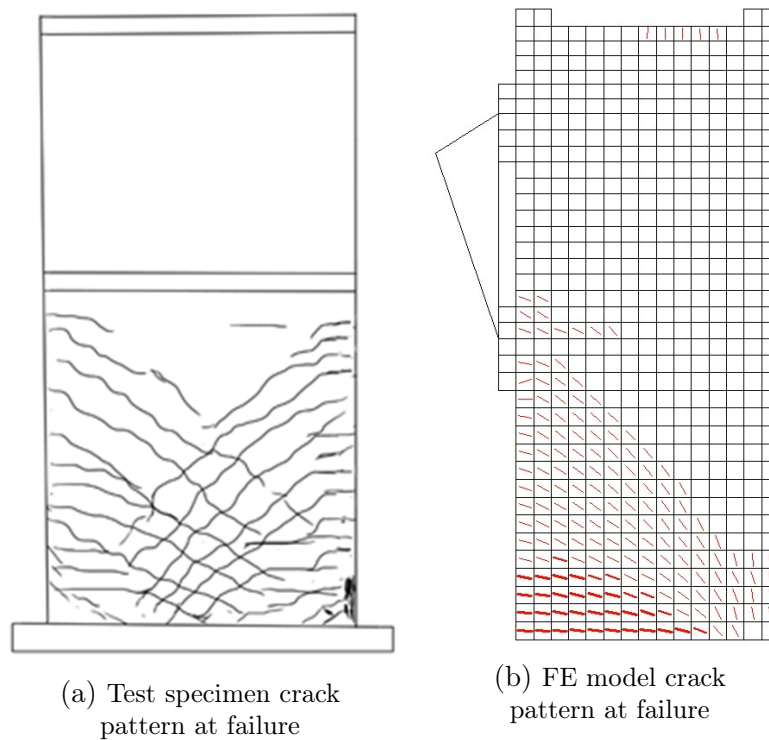
Figure 4.9: SW3 force-displacement curve

The force-displacement curve of the numerical simulation fits the observed behaviour of the test specimen quite well. The peak resistance of 288.6 kN is only 0.6% lower than the measured one of 290.3 kN, it is worth to note that the imposed force model gave a peak resistance of 290.5 kN. Displacement capacity is here quite similar to the laboratory test results. After a top displacement of 55 mm, the results started oscillating before a sudden resistance degradation. Resistance is then more accurately predicted than the ductility which is overestimated by about 20% which may be related to the additional truss structure.

### 4.3.2 Crack pattern and failure mode

The crack pattern is fairly well predicted by the FE simulation. However, strain seems to concentrate in the bottom left region of the wall, which was not reported in the test observations.

The observed failure mechanism of the test specimen is said to be a combination of flexure and shear. The shear vulnerability of the wall actually seems quite low, and the failure is mainly due to the crushing of the compressed concrete before with buckling of the reinforcements. The last reliable data point of the analysis however did not show any



concrete crushing as the maximal compressive strain of the concrete was no greater than 3.6%.

## 4.4 Isakovic et al. - 2021

A study was conducted to investigate the coupling of shear walls by slabs [17]. A half-scale setup of four walls linked by the slabs of three stories was built and tested on a shake table. The paper claims that the frame effect due to the slab coupling is responsible for more than 50% of the total bending moment resistance, making it a coupled system by Eurocode 8 standards.

### 4.4.1 Modelling of the coupling slab

The slab being the coupling element, its bending stiffness as well as its shear resistance will dictate the magnitude of the coupling forces and the importance of the frame behaviour of the system. It is therefore important to model those slab as well as possible. Given its flexural behaviour and small thickness dimension compared to the mesh size of the wall, discrete reinforcements were used instead of smeared reinforcement as it is for the walls. The shear resistance of the slab is produced mainly by the dowel action of the longitudinal reinforcements, and they are rarely fitted with shear reinforcements.

A 3 meters wide and 80 mm thick slab matching the dimensions of the coupling test specimen was also numerically tested by itself. Different mesh element dimensions were

compared in the following model, which tested the slab to the shear and bending it would experience as a coupling element.

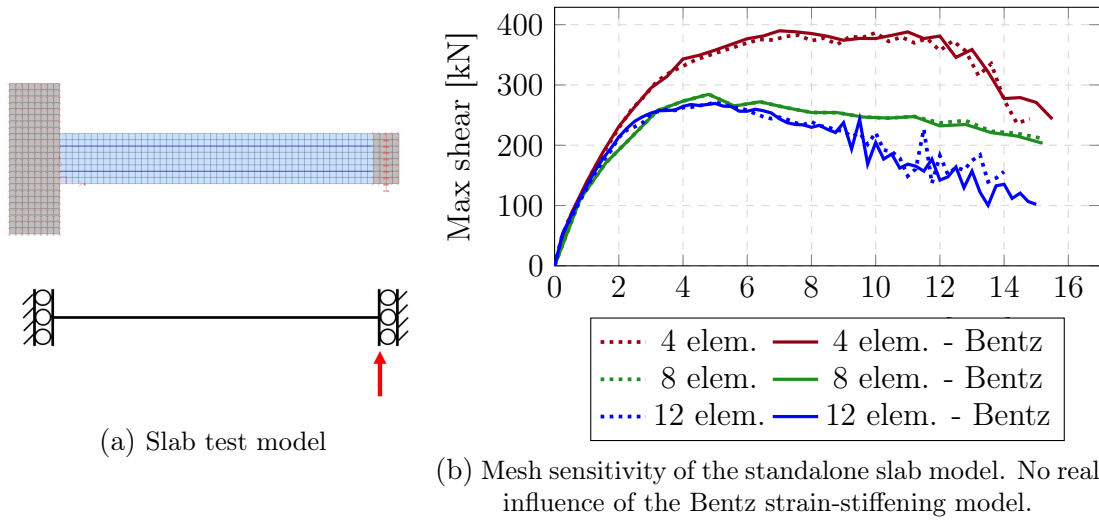


Figure 4.11: Slab shear test with different FE sizes and with or without the strain-stiffening model from Bentz.

It can be seen that for 8 and 12 elements along the thickness of the slab, the same resistance is achieved with a more important softening for the 12 element mesh. The more coarse 4 elements mesh displays almost 45% increase in shear resistance of the slab. The 8 element mesh will therefore be used in the different complete models. Slabs in their coupling configuration will not exhibit shear resistance as high as the ones of Figure 4.11 since they will be a lot less axially loaded in compression.

The strain-stiffening model does not appear to have a great influence on the shear-displacement of the slab. However it does, again, causes strain localization in the complete model and will therefore not be used.

#### 4.4.2 Pushover analysis and force-displacement curve

Walls had a length of 0.75 m and a thickness of 100 mm. All stories were 1.5 m high and the slab had a surface of 3 by 3 m. The wall had transverse and longitudinal reinforcement ratios of  $\rho_t = 0.75\%$  and  $\rho_l = 0.8\%$ . The walls were only subjected to gravity loads and no additional axial loading was imposed, the axial load ratio was therefore below 1%.

Vector2 being a 2D FE software, it was not possible to represent the four walls. The configuration has been adapted simply by doubling the thickness of the wall in the model to 200 mm. This model was then submitted to an imposed force pushover analysis. Shake table testing is not directly correlated to this kind of analysis, but it should give a good indication on the ability to capture the frame effect induced by the slabs. Linearly distributed load was applied with the following distribution : first floor = 17%  $V_b$  ; second floor : 33%  $V_b$  ; third floor : 50%  $V_b$ .

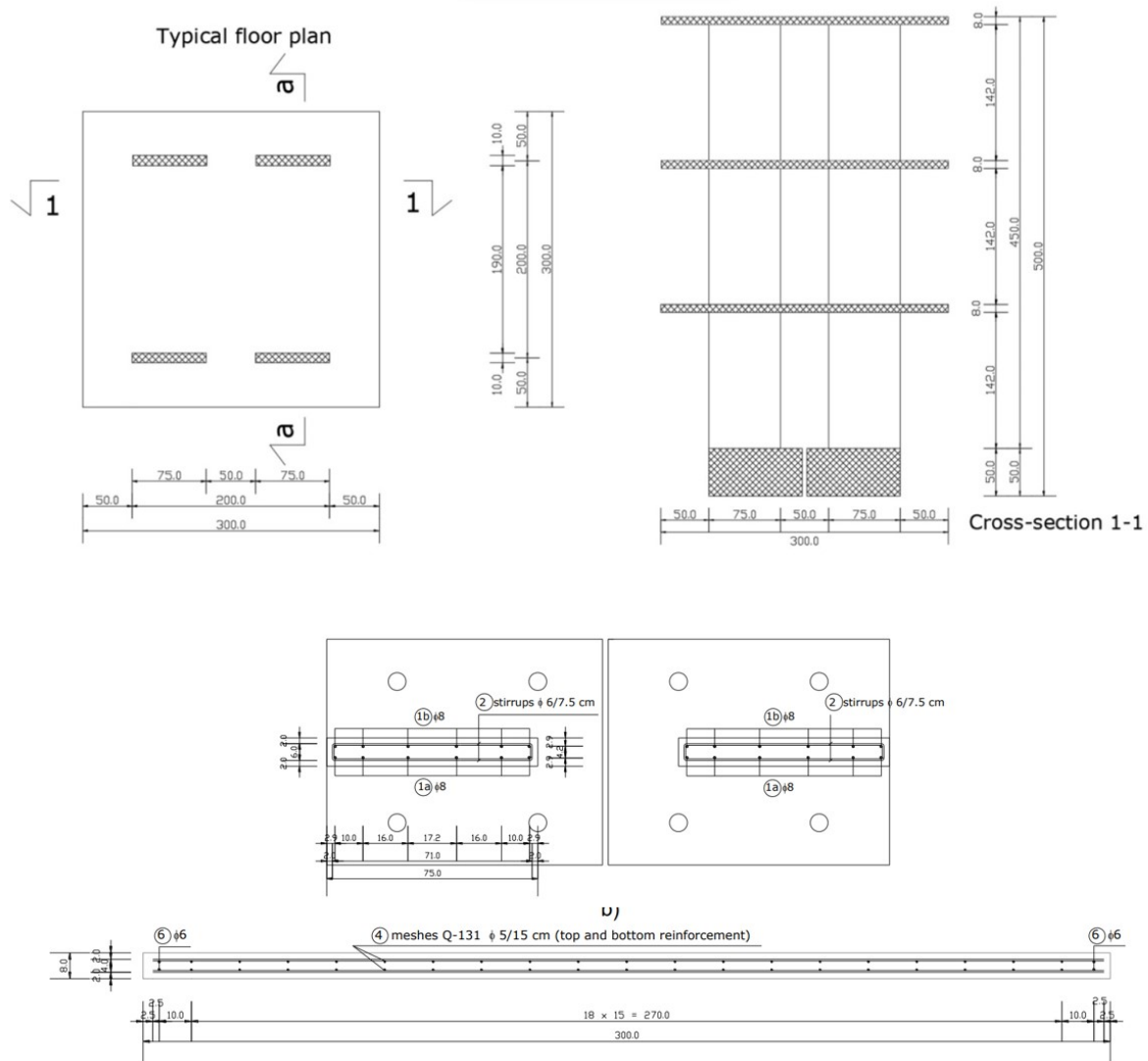


Figure 4.12: Test specimen dimensions and reinforcements of the wall

The slab on the exterior sides of walls is not modelled as it does not have any structural role. Its width is taken as 3 m to fit the test specimen, but an effective width of 900 mm computed using Eq. 2.14 is also analysed.

The imposed force analysis does not allow capturing the post-peak behaviour as mentioned previously. Figure 4.13 shows that the envelope of the shake table test results and the FE prediction does not seem to really fit. While the difference in maximum resistance is only -8% between the 182 kN of the test and the 188 kN of the FE model, the stiffness of the 3 m effective width model is higher than the test specimen. This can be due to the load distribution, which is certainly slightly different from the linear loading applied in the FE simulation.

The 900 mm slab effective width model is also underestimating the resistance by around 40%. This may be caused by the fact that two set of parallel walls were tested, and they therefore managed to mobilize a larger part of the slab in bending than if only two walls

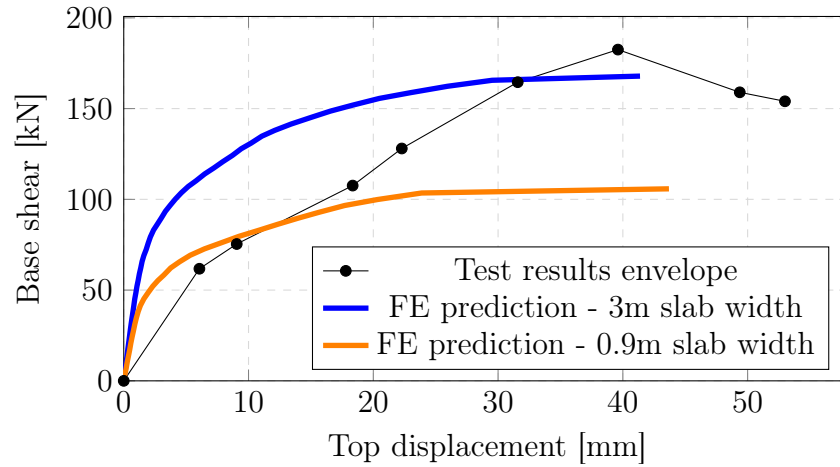


Figure 4.13: Force-displacement curves of shake table test and FE predictions for different effective width

had been considered.

#### 4.4.3 Coupling action

By analysing the efforts at the base of the wall, it is possible to detect the influence of the coupling slabs on the resistance of the wall using the equilibrium equation of Figure 2.11. In the following table, the "Wall 1" refers to the left wall which will be in tension due to the pushover and "Wall 2" refers to the right, compression wall.

|                   | Width = 3 m |        | Width = 0.9m |        |
|-------------------|-------------|--------|--------------|--------|
|                   | Wall 1      | Wall 2 | Wall 1       | Wall 2 |
| $N_g$ [kN]        | -40         |        |              |        |
| $N_{tot}$ [kN]    | 187         | -271   | 11           | -97    |
| $N_{frame}$ [kN]  | 227         | -231   | 51           | -57    |
| $M_b$ [kNm]       | 66          | 216    | 124          | 163    |
| $V_b$ [kN]        | 44          | 124    | 37.5         | 68     |
| $M_{WALLS}$ [kNm] | 282         |        | 287          |        |
|                   | <b>33%</b>  |        | <b>68%</b>   |        |
| $M_{FRAME}$ [kNm] | 572         |        | 135          |        |
|                   | <b>67%</b>  |        | <b>32%</b>   |        |
| $M_{TOT}$ [kNm]   | 854         |        | 422          |        |

Table 4.1: Base wall efforts and frame effect highlight

For the larger width model, the frame effect is really important as it represents around

67% of the total moment resistance. The additional axial load caused by the coupling slabs amounts to around 230 kN total, with the first floor slab contributing 86 kN, the second 67 kN and the third one 75 kN. This magnitude of coupling effect is significantly higher than the value reported in the test observations, even if the two configurations are not entirely comparable.

The second model using the effective width shows a weaker coupling, but nonetheless a significant amount of the resisting moment is caused by the coupling through the slabs.

## 4.5 Limitations

### 4.5.1 Localization and mesh sensitivity

Strain localization is actually a real phenomenon observed in material mechanics, which can see the strain growing in a specified location of a specimen submitted to an external loading. The strain is therefore restricted to a finite region and failure occurs in this region, and the phenomenon is dependent on the test specimen size [36]. Numerical localization can also occur in finite element simulations due to the issue of capturing the post peak strain-softening behaviour of a material.

Strain-softening and tensile cracking induce an inversion of slope of the stress-strain curve of a material. Meaning that the cracking or softening dissipates energy through deformation while the stress of the element decreases.

If a weaker or more solicited element starts cracking, the neighbouring element can unload, and the strain will then localize in the cracked element. A webinar hosted by Massimo Petracca on Asdea Software Technology Youtube channel approached this issue with the following analogy.

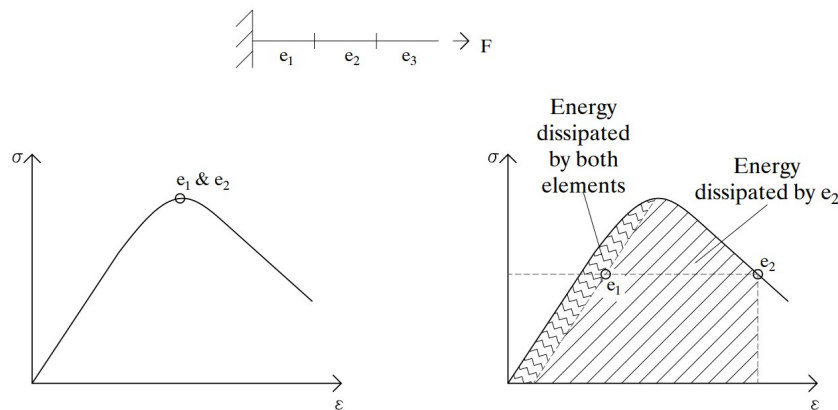


Figure 4.14: Strain localization illustration

Elements placed in series are subjected to axial loading. The element 2 is weaker and enters the strain softening downward curve first. To keep equal stresses in all elements,

element 1 unloads. The majority of the energy dissipation, depicted on the graph by the area below the stress-strain curve, is mainly dissipated by e2 only.

The energy dissipated by element 2 then depends on its size. Therefore larger elements will be able to dissipate more energy and exhibit a more ductile behaviour. When refining the finite element mesh using smaller elements they will be able to dissipate less energy, localization will be quicker to occur, and finally the models will exhibit less ductility.

This causes some finite element models to be highly sensitive to the mesh elements dimensions and can cause results to be deemed untrustworthy [33].

Such localization actually occurred in numerous models using the Modified Bentz strain hardening model, as shown in Figure 4.15. The model for the VK3 wall showed localization of strain in the web just past the peak of the force-displacement curve. It is mainly why this model was not used and strain hardening of concrete was not considered.

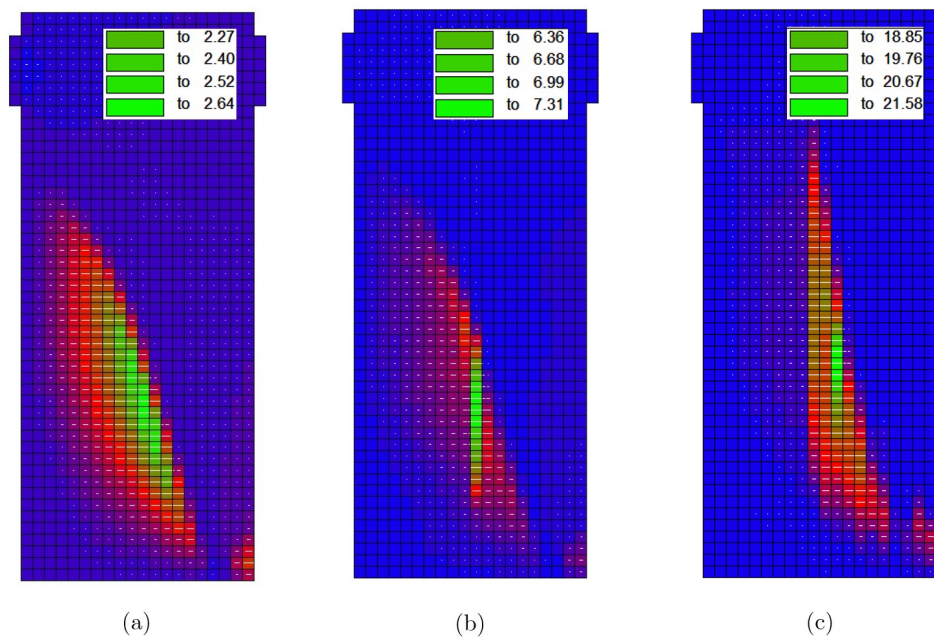


Figure 4.15: Horizontal strain for VK3 model at (a) load step 25 (14mm top displ.), (b) load step 27 (15mm top displ.), (c) load step 54 (29mm top displ.)

Different methods exist to deal with strain localization (regularization, fracture energy, ...) but they were not implemented here.

### 4.5.2 Dependent imposed displacements

The truss system to impose displacements at two different positions while keeping their applied force proportional is not without effect on the simulation and the results.

It complicates the task of the algorithm to reach convergence between the different load steps as the stiffness matrix of the model is more complicated, giving less precision to the final output. The upper part of the wall can not deform freely. The truss being extremely rigid, the left side of the second story can not elongate as it would without this additional structure. It will actually induce an opposite secondary curvature. It has been shown that difference with the actual applied forces is non-existent up to the peak resistance 4.9 but in the inelastic regime, the same can not be said. Finally the actual distribution of the imposed displacement is defined by the geometry of the structure but the stiffness of the wall at both application point being different, this distribution can be slightly altered.

As the parametric study of the next chapter includes this setup structure in both models analysing the different parameters, its impact is theoretically the same for all the wall configurations tested. The negative impact of this truss system on the results is thus estimated to be tolerable.

### 4.5.3 Software and modelling limitations

The models do not take out of plane instability nor spalling. As mentioned, foundation blocks were not modelled, resulting in a possible underestimation of ductility since no strain penetration is allowed.

For the coupling slabs, the FE being 2D, it can not capture the stress distribution in the plane of the slab and can not assess its real effective width nor can it capture the potential puncture shear of the slab.

## Chapter 5

# Stirrup effect : modelling and parametric analysis

This chapter aims to investigate more deeply into the "stirrup effect" introduced in Section 2.4.3. To this end, numerous FE models have been built to try to capture this phenomenon and how it can affect the behaviour of reinforced concrete shear walls.

The reference wall configuration is first presented, and the subsequent parametric analysis and its results are hereafter described and discussed.

### 5.1 Reference model

The reference model of the parametric analysis is partly based on the VK3 wall from [12] presented in Section ???. As the latter was actually a bridge pier, on which no floor diaphragm could realistically be added, its dimensions were modified to create a "full-scale" shear wall in the following configuration.

The transverse reinforcement ratio  $\rho_t$  is purposely low for this model as it should allow for a more shear-critical behaviour which is supposed to engage the floor diaphragm in the stirrup effect interaction. It would not satisfy current Eurocode standards, as the minimal amount of transversal reinforcements should here be  $0.25 \cdot 1.23\% = 0.3075\%$ . This therefore would not be relatable to currently designed walls, but would completely fit the configuration of already existing walls.

As mentioned in 3.5, the threshold value for the steel reinforcements ultimate strain is taken as  $0.6 \cdot \epsilon_{su} = 9\%$ .

Floor diaphragm width (out of the wall's plane) is set to 3.5 m. Considering that the whole floor slab will not contribute to the stirrup effect, the effective width is taken as half of the wall length on each side. After adding the wall thickness of 350 mm and rounding it up, the slab width is defined as 3.5 m

|                                |                    |       |     |
|--------------------------------|--------------------|-------|-----|
| <b>Wall dimensions</b>         |                    |       |     |
| First floor height             | $h_1$              | 3.1   | m   |
| Second floor height            | $h_2$              | 3     | m   |
| Length                         | $l_w$              | 3     | m   |
| Thickness                      | $t$                | 350   | mm  |
| Aspect ratio                   | $L_s/l_w$          | 1.85  | /   |
| <b>Concrete</b>                |                    |       |     |
| Compression strength           | $f'_c$             | 34    | MPa |
| Ultimate strain                | $\epsilon_{cu,c*}$ | 0.53  | %   |
| <b>Reinforcements</b>          |                    |       |     |
| Vertical reinforcement ratio   | $\rho_l$           | 1.23  | %   |
| Transverse reinforcement ratio | $\rho_t$           | 0.185 | %   |
| Ultimate strain                | $\epsilon_{su}$    | 15    | %   |
| Axial load ratio               | $N/A_c f_c$        | 10    | %   |
| <b>Floor diaphragm</b>         |                    |       |     |
| Width                          | $w_{fd}$           | 3.5   | m   |
| Thickness                      | $t_{fd}$           | 200   | mm  |

Table 5.1: Reference wall properties

This wall was modelled with 15 elements along its length. This mesh refinement is sufficient to predict the behaviour of the wall, as showed in the validation chapter. The amount of FE constituting the model is also limited by the high computational power requested by the additional truss structure. The section at the location of the diaphragm is then enlarged on a second model to consider the floor and its restraining effect.

### 5.1.1 Force-displacement curve - ductility

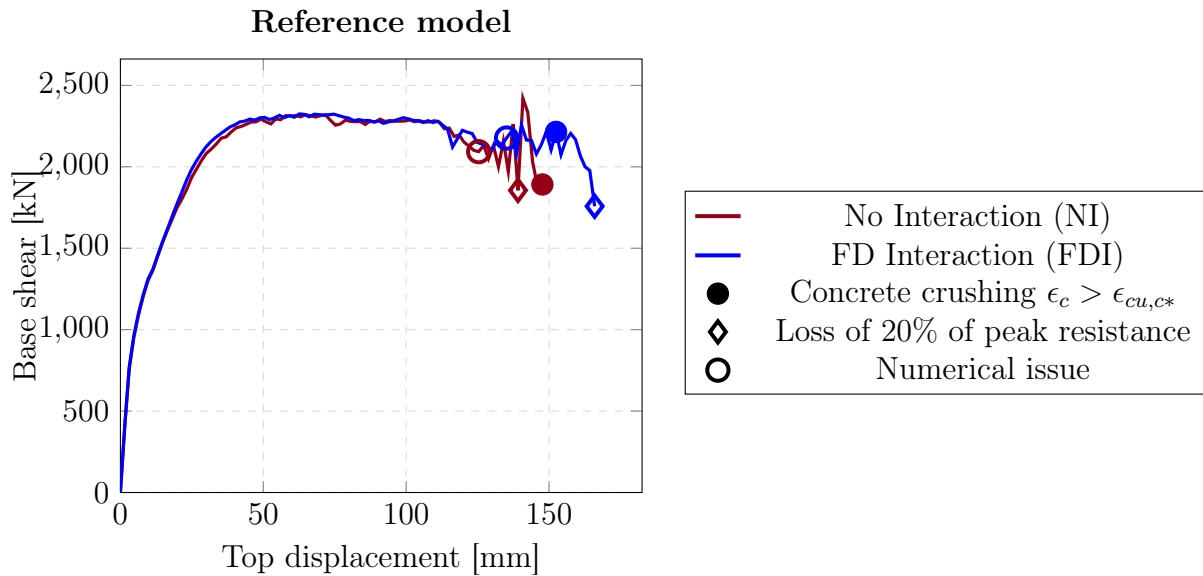


Figure 5.1: Force-displacement curve for the reference model

Force-displacement curve prediction is then compared for the models with and without the

floor diaphragm. Figure 5.1 shows that the two curves are highly similar. They actually present almost the exact same values up to a top displacement of around 107 mm. The different criteria mentioned in Section 3.5 are indicated. Numerical oscillations occurred respectively after 125 and 135 mm top displacement without and with diaphragm respectively. The results obtained after could therefore be considered somewhat unreliable. Concrete crushing at the toe is observed after a 147 mm top displacement for the simple model and after 152 mm for the model considering the stirrup effect, which is relatively insignificant. The resistance threshold of 80% of the peak resistance was reached for a top displacement of 139 mm for the model without diaphragm and 165 mm for the one with diaphragm, an increase of 30 mm by which is to be considered carefully as other criteria were also met.

For the reference configuration, the pre-peak behaviours are identical and the post-peak are relatively similar for both models. The different failing criteria occurrences are really close and do not allow to clearly establish a difference of displacement capacity between the two configurations. Therefore, the impact of the stirrup effect in this case can be considered as non-existent regarding the ductility of the wall.

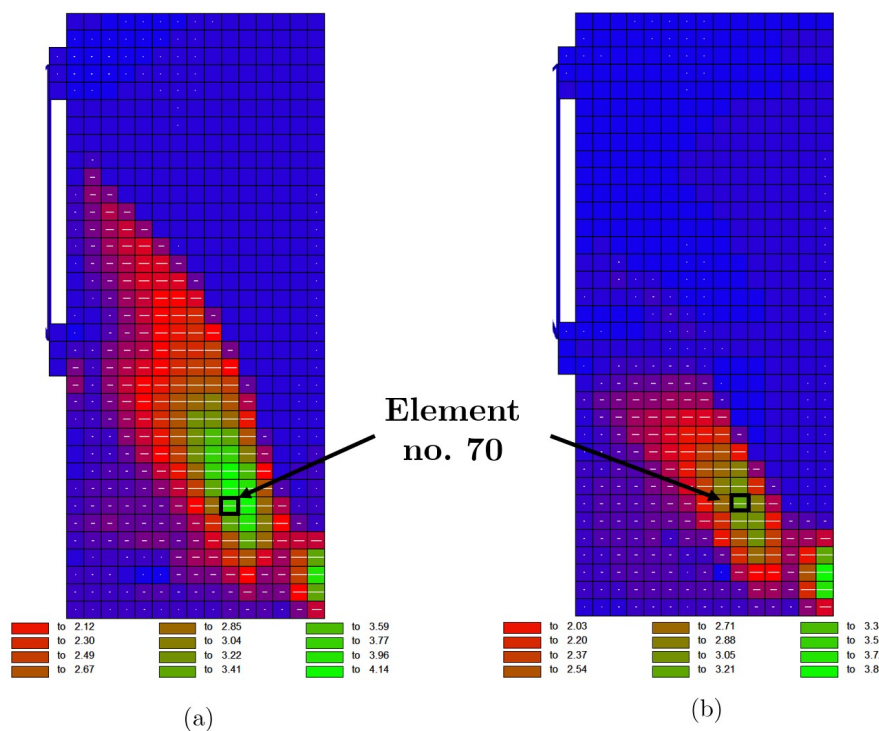


Figure 5.2: Horizontal strains of simple model (a) and model including a floor diaphragm (b) at peak resistance

Figures 5.2 and 5.3 show that horizontal strains distribution and magnitude is indeed altered by the floor diaphragm modelling. Some elements are subject to up to 25% less horizontal strain when the floor diaphragm is considered. This however does not seem to really affect the behaviour or the failure mode.

Models not including the floor diaphragm will hereafter be denoted as "No Interaction

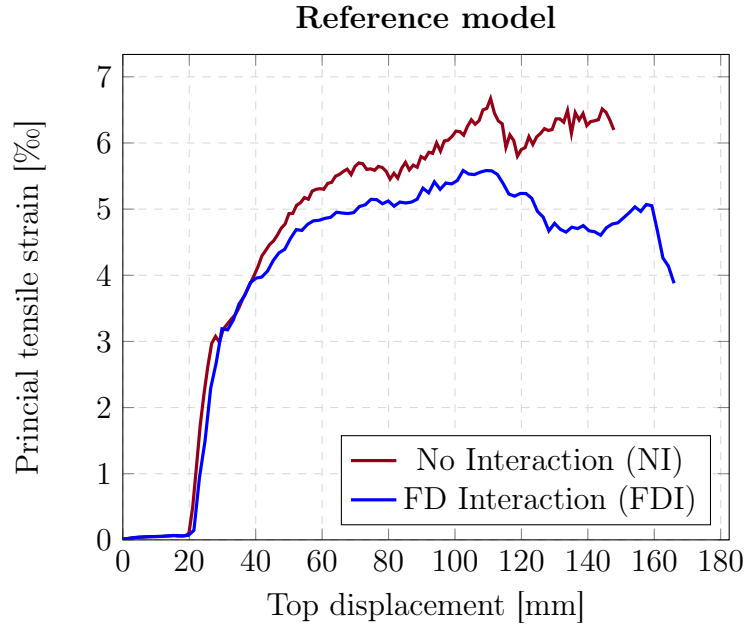


Figure 5.3: Tensile strain of element no. 70 in both configuration

models" or NI models whereas models which include the floor diaphragm will be referred to as "Floor Diaphragm Interaction" or FDI models for clarity.

## 5.2 Parametric analysis

### 5.2.1 Transversal reinforcement ratio

Different values of transverse reinforcement are selected to study their impact on a possible stirrup effect of the floor diaphragm on the wall. All the other parameters remain unchanged. For each value of  $\rho_t$ , a different value threshold value for  $\epsilon_{cu,c*}$  is adopted according to Section 3.5. Reducing the reinforcement ratio should decrease the shear resistance of the section and cause a shear failure to occur, which would certainly engage action from the floor diaphragm restraint.

$$\rho_t = 0.10\%$$

Force displacement curves displayed on Figure 5.4 are quite close until the top displacement of 90 mm at which differences start to show. The threshold value for the concrete ultimate strain  $\epsilon_{cu,c*}$  given that it is somewhat confined and that the wall has a transverse reinforcement ratio of  $\rho_t = 0.10\%$  is defined at 0.47%. This value is exceeded for a top displacement of 119 and 145 mm for the NI and FDI models respectively, an increase of 22%. The commonly used criterion of lateral load resistance degradation is met after 121 mm for the NI simulation and after 145 mm for the FDI simulation, an increase of 20%. No numerical issues were noted for this model. As can be seen on Figure 5.4, horizontal strain is reduced by the presence of the floor diaphragm and for element no. 70, this difference can rise to more than 50%.

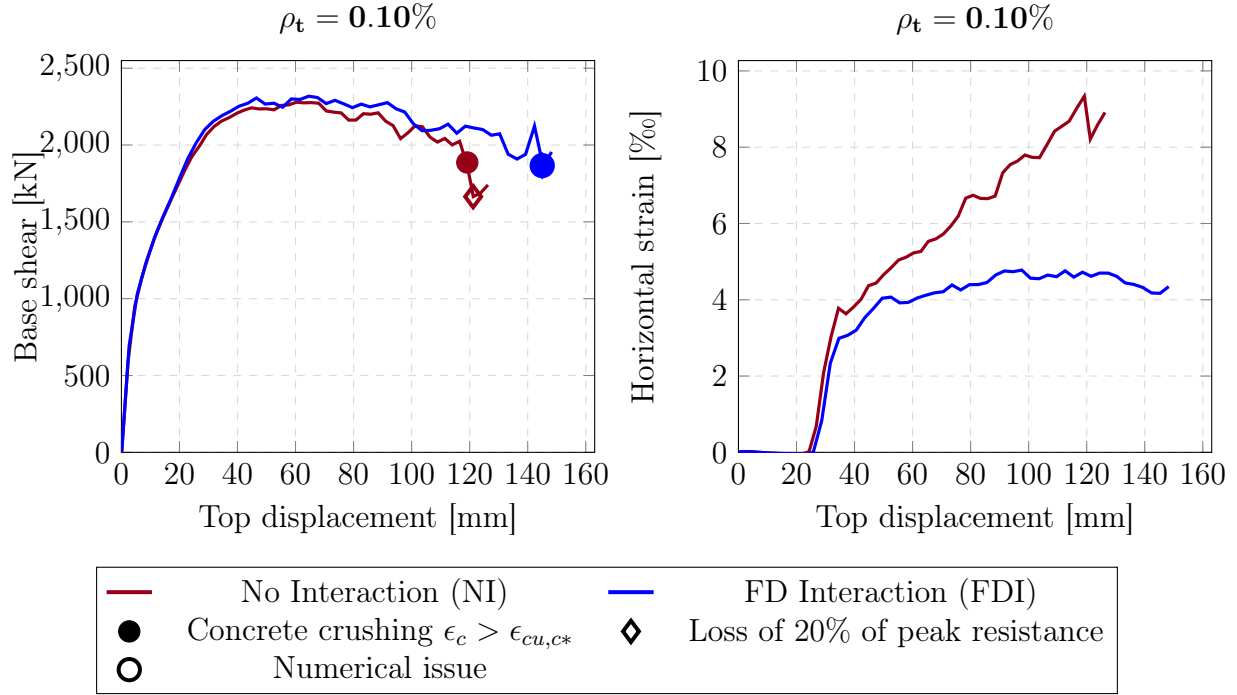


Figure 5.4: Force displacement curves and horizontal strain of element no. 70 for  $\rho_t = 0.10\%$

All the ductility limiting criteria seem to show a slight augmentation. This could mean that the ductility of the wall is indeed increased by the presence and modelling of the floor diaphragm and its restraint on the wall. However those values remain really close and as their exact value rely on different somewhat variable parameters (20% threshold value,  $\epsilon_{cu,c*}$ , load step, mesh size,...), it is not safe to conclude that the floor diaphragm has an clear influence in this configuration.

$\rho_t = 0.15\%$

Figure 5.5 shows how both force-displacement curves are highly similar up to the top displacement of 100 mm where they seem to follow different paths. Transverse reinforcement ratio  $\rho_t$  of 0.15% translates to a concrete ultimate strain of 0.51%. Both models go beyond this value. The NI model shows concrete crushing at the toe for a top displacement of 104 mm whereas the FDI model shows the same crushing at 157 mm, a difference of 51%. Both models present a combine shear-flexure failure mechanism and fail due to concrete crushing. Lateral load resistance degradation is reached in both models for 130 and 162 mm top displacement for the NI and FDI simulations respectively, an increase of 25%.

Finally, numerical issues in the form of oscillations are noted in both models after 174 mm top displacement for model NI and 162 mm for model FDI, a difference of 6%. Horizontal strain of element 70 displayed on Figure 5.5 shows how the floor diaphragm reduces the magnitude of strain in the bottom part of the wall. The reduction from the NI model can be as high as 47% for this precise element.

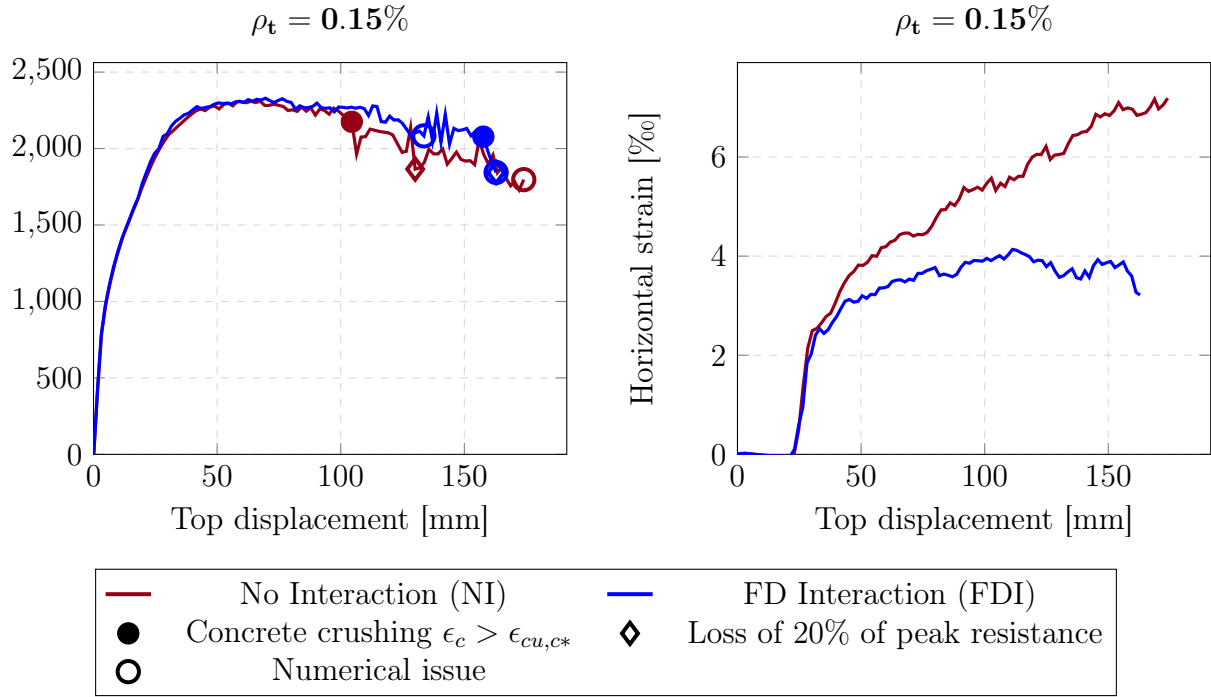


Figure 5.5: Force displacement curves and horizontal strain of element no. 70 for  $\rho_t = 0.15\%$

Concrete crushing and resistance degradation seem to both have been delayed by 51 and 25% respectively. One could conclude that the ductility is therefore increased by the presence of the floor diaphragm. However, the large increase regarding the concrete crushing is to be cautiously considered, as it does not make physical sense for it to happen earlier in the NI model than with a transverse reinforcement ratio of  $\rho_t = 0.10\%$ .

$$\rho_t = 0.25\%$$

Force displacement curves displayed on Figure 5.6 are almost identical, apart from the oscillations around 140 mm top displacement. The threshold value for the concrete ultimate strain  $\epsilon_{cu,c*}$  given that it is somewhat confined and that the wall has a transverse reinforcement ratio of  $\rho_t = 0.25\%$  is defined at 0.57%. Both models go beyond this value. The NI model shows concrete crushing at the toe for a top displacement of 150 mm whereas the FDI model shows the same crushing at 177 mm, a difference of 18%. Both models present a combine shear-flexure failure mechanism and fail due to concrete crushing. Lateral load resistance degradation is reached in both models for 192 and 183 mm top displacement for the NI and FDI simulations respectively, a difference of -5%.

Both simulations started showing oscillations for 137 mm top displacement for model NI and 148 mm for model FDI. As can be seen on Figure 5.6, horizontal strain is reduced by the presence of the floor diaphragm and for element no. 70, this difference can rise up to more than 30%.

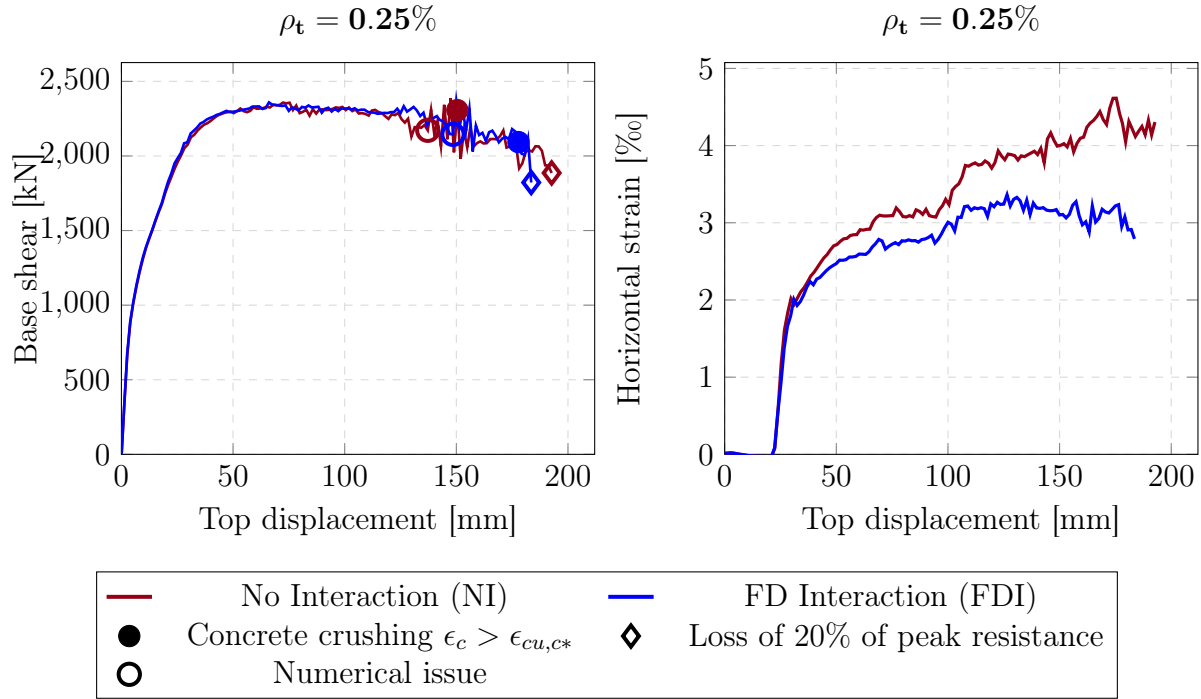


Figure 5.6: Force displacement curves and horizontal strain of element no. 70 for  $\rho_t = 0.25\%$

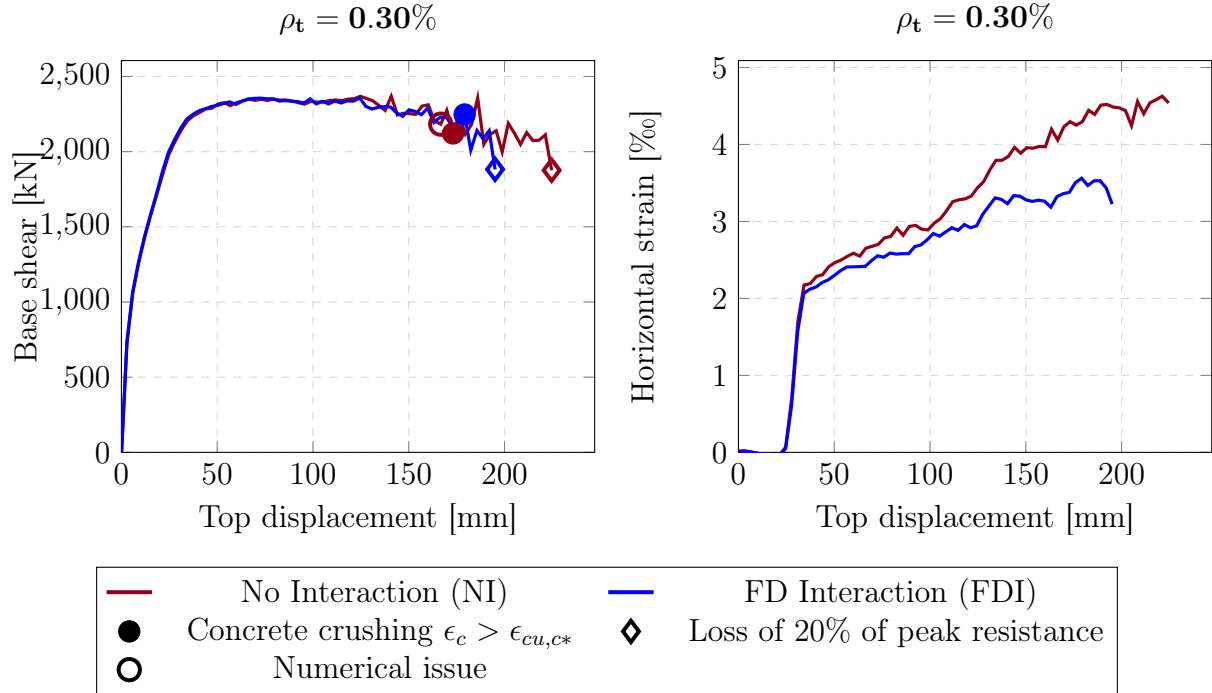
The results show a slight increase in the displacement capacity regarding the concrete crushing when considering the stirrup effect. Load resisting capacity however seem to decrease, which is somewhat unexpected. The stirrup effect does not seem to affect the behaviour of the wall in such a configuration.

$$\rho_t = 0.30\%$$

As presented on Figure 5.7, both force-displacement curves are almost identical. Transverse reinforcement ratio  $\rho_t$  of 0.15% translates to a concrete ultimate strain of 0.51%. This value is exceeded for a top displacement of 173 and 179 mm for the NI and FDI models respectively, an increase of 3%. This concrete crushing is therefore the cause of the combined shear-flexure failure of both models. The lateral load resistance degradation criterion is met after 224 mm for the NI simulation and after 214 mm for the FDI simulation, a difference of -13%.

The NI model showed some considerable oscillations after the top displacement of 166 mm is reached. Horizontal strain of element 70 displayed in Figure 5.7 shows how the floor diaphragm reduces the magnitude of strain in the bottom part of the wall. The reduction from the NI model can be as high as 22% for this precise element.

Increasing the transverse reinforcement ratio to 0.30% has proved to reduce the gap between the predicted displacement capacity as expected. From the observed results, the stirrup effect does not seem to affect the behaviour of the wall under this configuration. Horizontal strain reduction is substantially weaker than for lower reinforcement ratios.

Figure 5.7: Force displacement curves and horizontal strain of element no. 70 for  $\rho_t = 0.30\%$ 

|                              | $\rho_t = 0.1\%$ | $\rho_t = 0.15\%$ | $\rho_t = 0.25\%$ | $\rho_t = 0.30\%$ |
|------------------------------|------------------|-------------------|-------------------|-------------------|
| Peak resistance              | +1%              | +2%               | +0%               | +0%               |
| <b>Displacement capacity</b> |                  |                   |                   |                   |
| - Concrete crushing          | +22%             | +51%              | +18%              | +3%               |
| - Resistance degradation     | +14%             | +25%              | -5%               | +0%               |

Table 5.2: Changes in resistance and displacement capacity for different transverse reinforcement ratios

As expected, the biggest impact of the floor diaphragm on the model was measured for a very low transverse reinforcement ratio. Increase of displacement capacity between 14 and 22%, depending on the criteria, was detected for  $\rho_t = 0.10\%$ . However, those values are highly sensitive to multiple modelling factors, and they can not be taken as exact values. Horizontal strain reduction is also weaker for higher transverse reinforcement ratio. The shear-flexure failure mode was unchanged when adding the floor diaphragm into the model.

### 5.2.2 Axial load ratio

Axial load ratio was then analysed as another parameter. This parameter can significantly impact the behaviour of shear walls, as it increases the bending resistance of the wall but not its shear resistance, allowing for shear failure to occur in the inelastic domain. Crushing of concrete is of course also more likely.

ALR = 5%

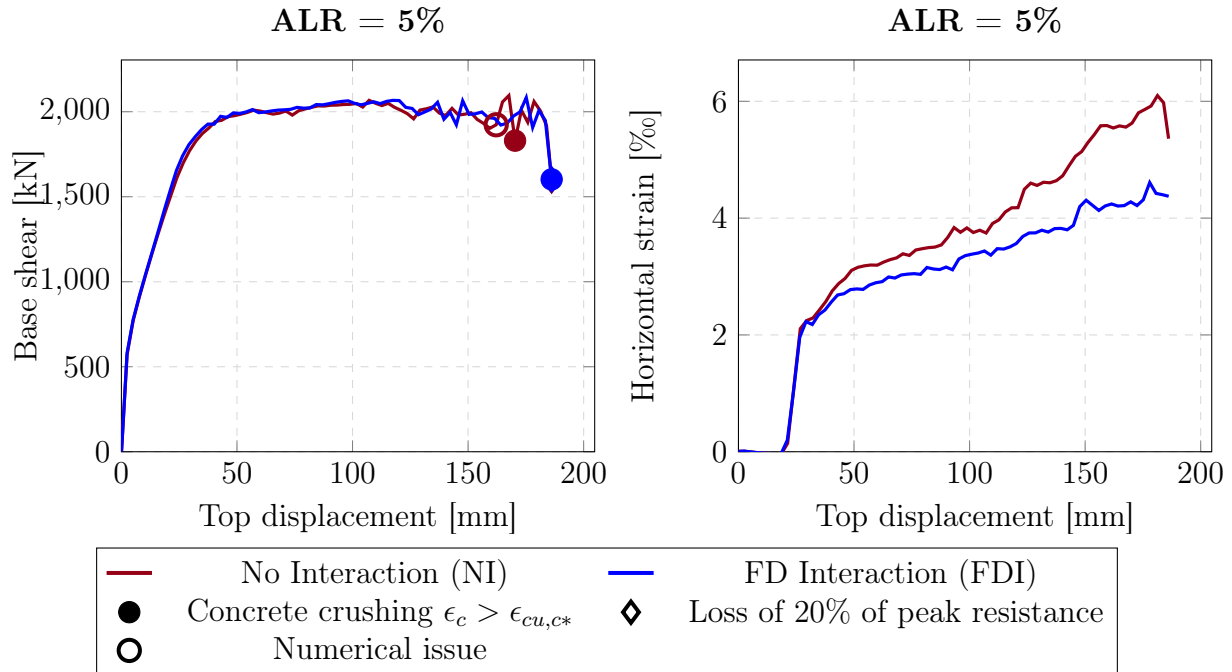


Figure 5.8: Force displacement curves and horizontal strain of element no. 70 for ALR of 5%

Figure 5.8 shows how both force-displacement curves are almost identical. Both models have the same shear-flexure combination failure mode caused mainly by concrete crushing. This concrete crushing at the toe is observed for a top displacement of 170 and 186 mm for the NI and FDI models respectively, an increase of 9%. Lateral load resistance degradation of 20% is reached in both models at the exact same top displacement of 186 mm.

The NI model exhibited some numerical issues in the form of oscillations from a top displacement of 162 mm. As can be seen on Figure 5.8, horizontal strain is reduced by the presence of the floor diaphragm and for element no. 70, this difference can rise up to more than 27%.

No major differences were observed between the NI and FDI models for an ALR of 5%. As expected, reduces the maximum bending resistance of the wall and the influence of the floor diaphragm is not apparent except in the horizontal strain reduction.

ALR = 15%

Force displacement curves displayed on Figure 5.9 are quite close until the top displacement of 117 mm at which differences start to show. The NI model shows concrete crushing at the toe for a top displacement of 142 mm whereas the FDI model shows the same crushing at 134 mm, a difference of -6%. Combined shear-flexure failure remains unchanged when adding the floor diaphragm. Both models exhibited lateral load resistance degradation for top displacement values almost identical. NI lost 20% of its resistance after 166

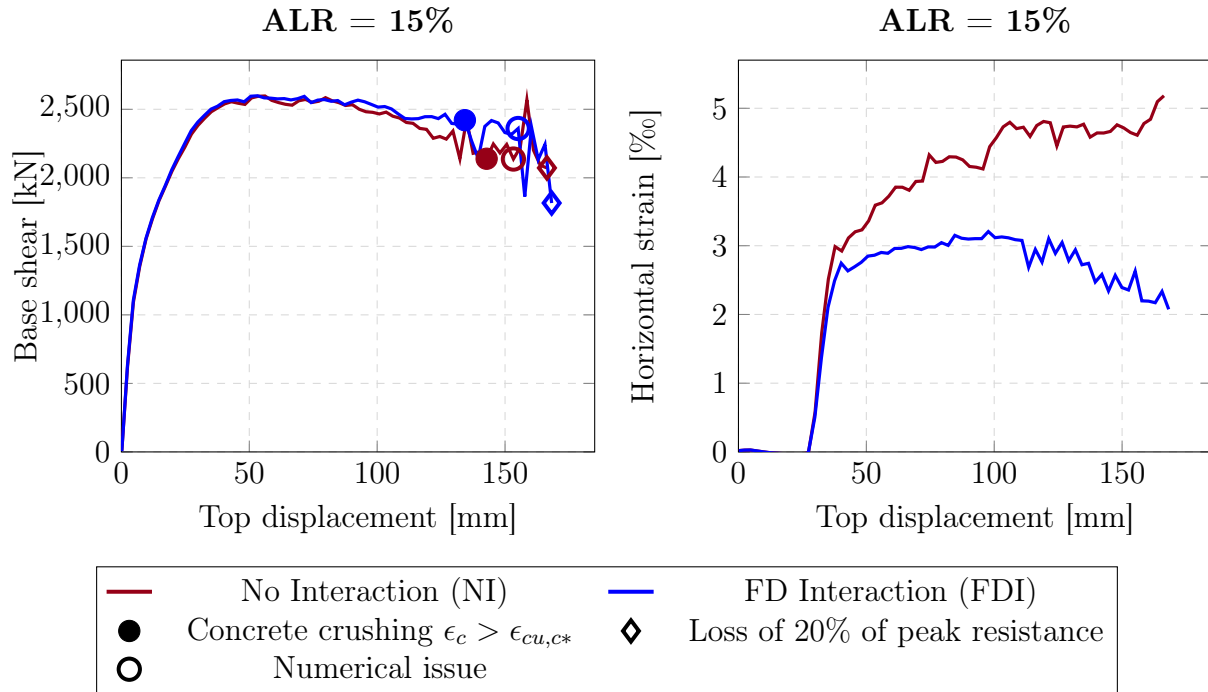


Figure 5.9: Force displacement curves and horizontal strain of element no. 70 for ALR of 15%

mm top displacement, and it was also the case for the FDI model after 168 mm. Finally, numerical issues in the form of abrupt change in resistance were noted in both models, after 153 mm top displacement for model NI and 155 mm for model FDI.

As can be seen on Figure 5.9, horizontal strain is reduced by the presence of the floor diaphragm and for element no. 70, this difference can rise up to more than 44%. Horizontal strain also seems to reduce after the top displacement of 100 mm for the FDI model. It is due to the fact that strain is concentrating at the toe of the wall.

The ductility of the wall does not appear to be influenced by the presence or not of the floor diaphragm. The different failure criteria beforehand are met for the same displacement capacity for both cases. Strain reduction however is very different when considering the floor diaphragm as it drops after a certain top displacement.

**ALR = 20%**

Force displacement curves displayed on Figure 5.10 are quite close until the top displacement of 90 mm at which small discrepancies begin to appear. Concrete crushing at the toe is observed for a top displacement of 126 and 137 mm for the NI and FDI models respectively, an increase of 8%. The commonly used criterion of lateral load resistance degradation is met after 121 mm for the NI simulation and after 131 mm for the FDI simulation, an increase of 9%. Small oscillations appeared in during the FDI simulation at a top displacement of 120 mm.

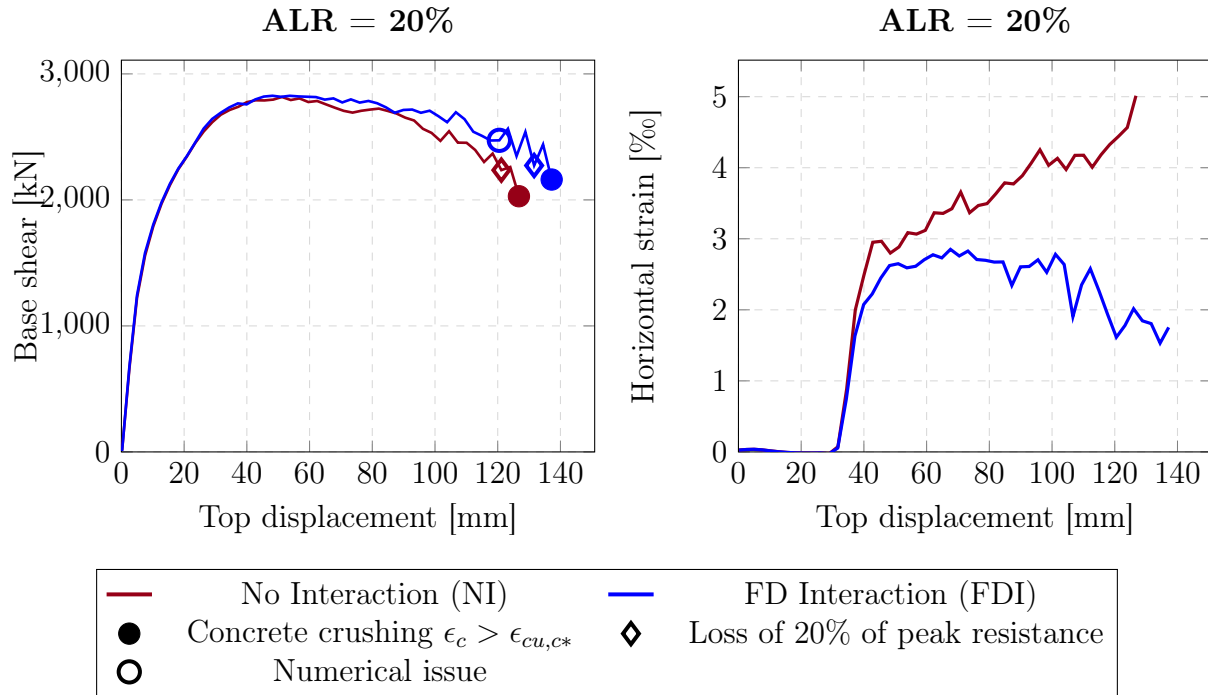


Figure 5.10: Force displacement curves and horizontal strain of element no. 70 for ALR of 20%

Figure 5.10 showcases the horizontal strain reduction induced by the floor diaphragm on the shear dominated region of the wall. For element 70, one of the most horizontally deformed elements, strain reduction in the FDI model can rise to 59%. Like for the ALR of 15%, horizontal strain in the FDI model seems to decrease after a top displacement of 100 mm.

Failure mode remains unchanged with the addition of the floor diaphragm to the model. Ductility of the wall slightly increases considering both the concrete crushing and the resistance loss by about 9%. This difference is too small to affirm that the floor diaphragm has a significant influence on the behaviour of the wall. Horizontal strain is reduced as expected.

### ALR = 25%

As presented on Figure 5.11, both force-displacement curves are almost identical. The NI model shows concrete crushing at the toe for a top displacement of 128 mm whereas the FDI model shows the same crushing at 122 mm, a difference of -5%. Lateral load resistance degradation is reached in both models for 119 and 131 mm top displacement for the NI and FDI simulations respectively, a difference of 9%.

A significant jump in the resistance of the NI model occurred for a top displacement of 125 mm which seems to be purely numerically caused. Horizontal strain of element 70 displayed on Figure 5.11 shows how the floor diaphragm reduces the magnitude of strain in the bottom part of the wall. The reduction from the NI model can be as high as 65%

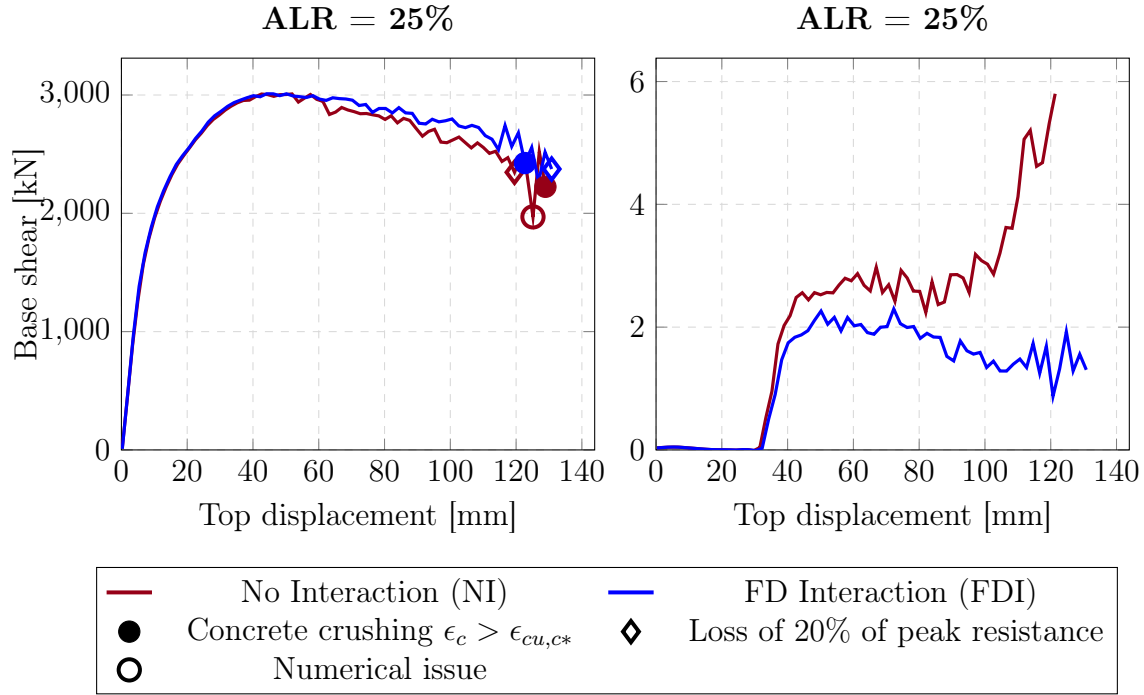


Figure 5.11: Force displacement curves and horizontal strain of element no. 70 for ALR of 25%

for this precise element.

Force-displacement curves for both models remained very similar, with the different ductility criteria being met at close displacement capacity. The failure mode remained the same. While horizontal strain was heavily reduced According to those results, floor diaphragm does not appear to influence the ductility of the wall or its behaviour.

|                                 | ALR = 5% | ALR = 15% | ALR = 20% | ALR = 25% |
|---------------------------------|----------|-----------|-----------|-----------|
| Peak resistance                 | +0%      | +0%       | +0%       | +0%       |
| <b>Displacement capacity</b>    |          |           |           |           |
| - <i>Concrete crushing</i>      | +9%      | -6%       | +8%       | -5%       |
| - <i>Resistance degradation</i> | +0%      | +1%       | +9%       | +9%       |

Table 5.3: Changes in resistance and displacement capacity for different axial load ratios

From the results of the five different values of axial loading ratio simulations, it does not appear to be a clear correlation between this parameter and the effect of the floor diaphragm on the behaviour of the tested wall configurations. Higher axial load ratios translated to more horizontal strain reduction, but the different failure criteria were generally met for the same displacement capacities. Failure modes also remained unchanged.

### 5.2.3 Vertical reinforcement ratio

Vertical reinforcement was also analysed in the scope of this study. Different ratio were tested to investigate their impact on a possible stirrup effect in different shear wall configurations.

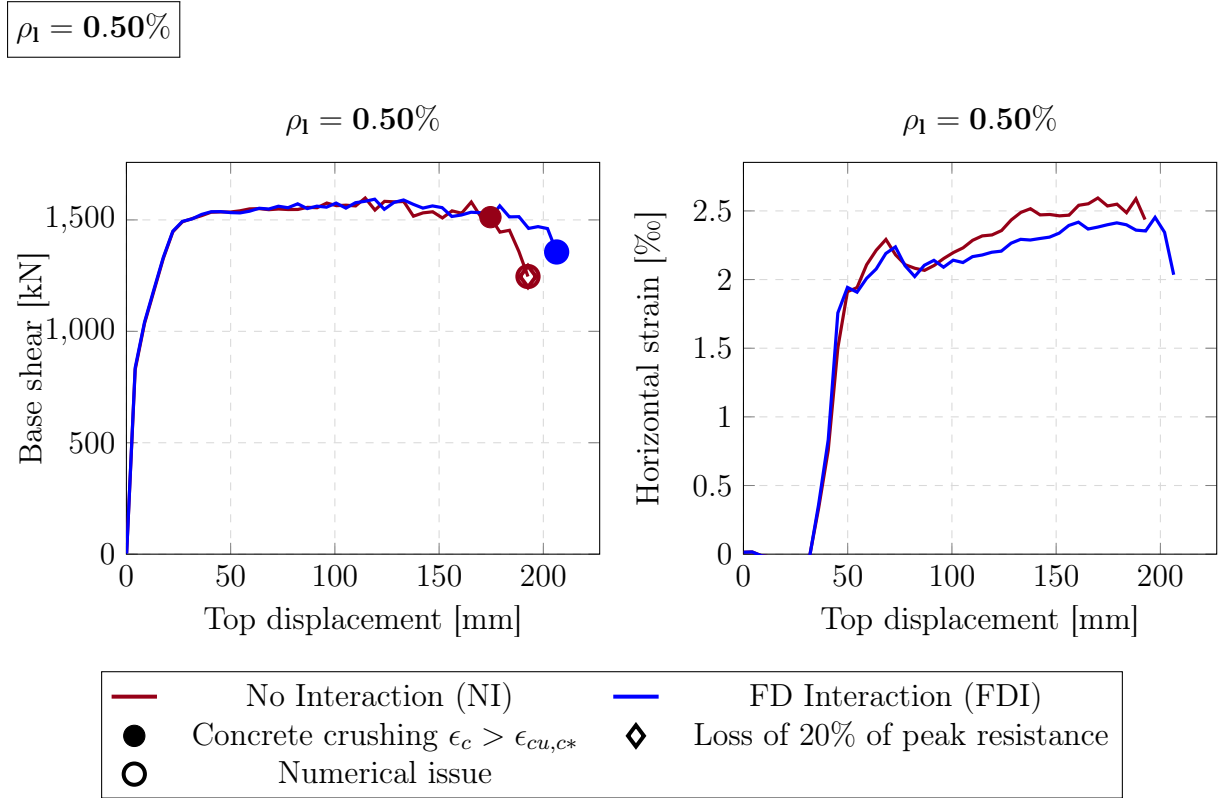


Figure 5.12: Force-displacement curves and horizontal strain of element np. 70 for  $\rho_l = 0.50\%$

Figure 5.12 shows how both force-displacement curves are highly similar up to the top displacement of 174 mm where they seem to follow different paths. Concrete crushing at the toe is observed for a top displacement of 174 and 206 mm for the NI and FDI models respectively, an increase of 18%. The shear-flexure failure mode is therefore not affected by the presence of the floor diaphragm. Lateral load resistance degradation is reached only in the NI models at 192 mm top displacement.

Compression strain localization occurred at the toe of both walls. While this physical phenomenon can indeed be observed in real testing conditions, it also may be due to numerical error. Results past this point have therefore been discarded. As can be seen on Figure 5.12, horizontal strain is slightly reduced by the presence of the floor diaphragm and for element no. 70, this difference can rise up to more than 8%.

An increase of the displacement capacity regarding the concrete crushing was observed in this simulation. One could deduce that the stirrup effect is indeed influencing the ductility of the wall. The rather small difference and the high sensitivity of the results however need to put this observation into perspective.

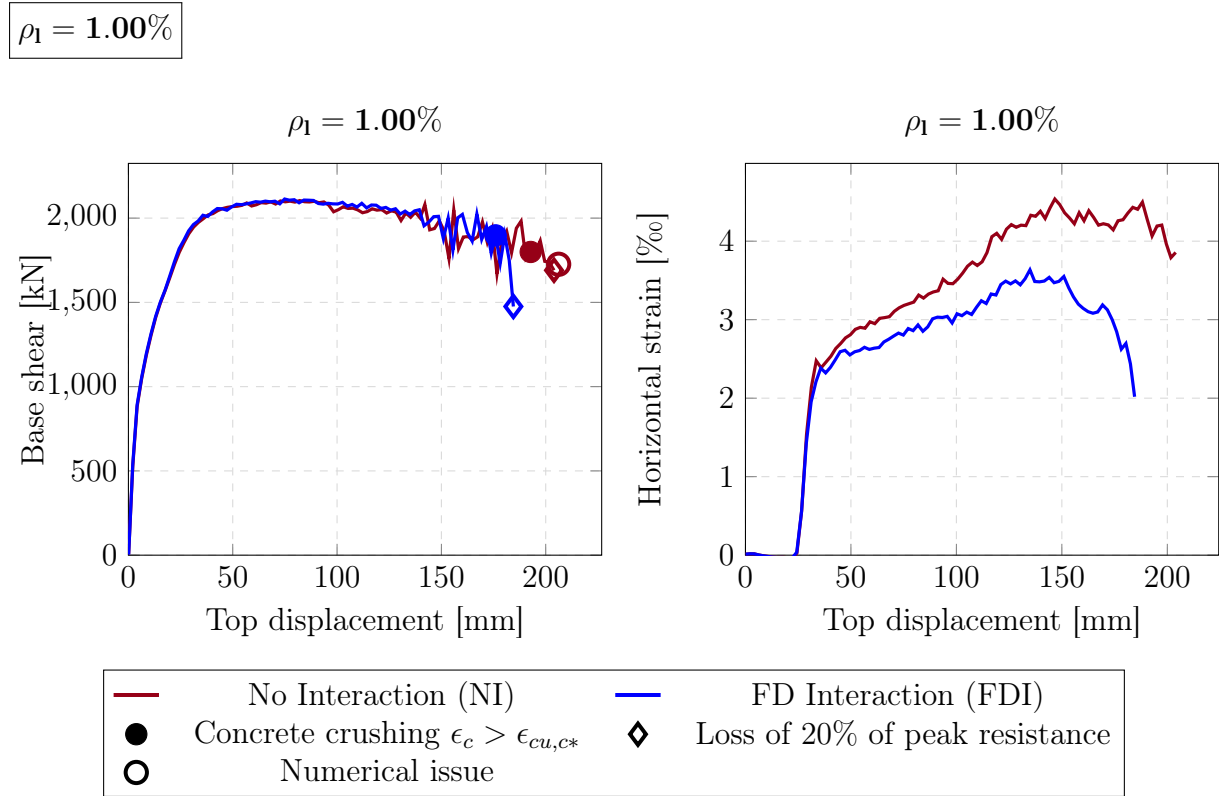


Figure 5.13: Force-displacement curves and horizontal strain of element np. 70 for  $\rho_l = 1.00\%$

Force displacement curves displayed on Figure 5.13 are quite close until the top displacement of 140 mm at which differences start to show. Localization at the base of the wall has caused the FDI model to actually exhibit concrete crushing at the toe for a top displacement of 176 mm while crushing of concrete was attained at 192 mm for the NI model, a difference of -9%. For the FDI model, resistance degradation criterion was met for a top displacement of 184 mm while the NI model maintained a resistance up to 203 mm of top displacement. Nonetheless, these results taking place after the strain localization they are unreliable.

Figure 5.13 highlights the horizontal strain reduction induced by the floor diaphragm on the shear dominated region of the wall. For element 70, one of the most horizontally deformed elements, strain reduction in the FDI model can rise to 44%. Huge drop in horizontal strain in the FDI model can also be noted after the top displacement of 150 mm.

Strain localization prevents from making a certain opinion about this simulation. It could reflect a real physical phenomenon, in this case the FDI model presents a lower ductility than the NI model, which is not really expected. Or it could be strictly numerical and therefore invalidate the results.

$\rho_1 = 2.00\%$

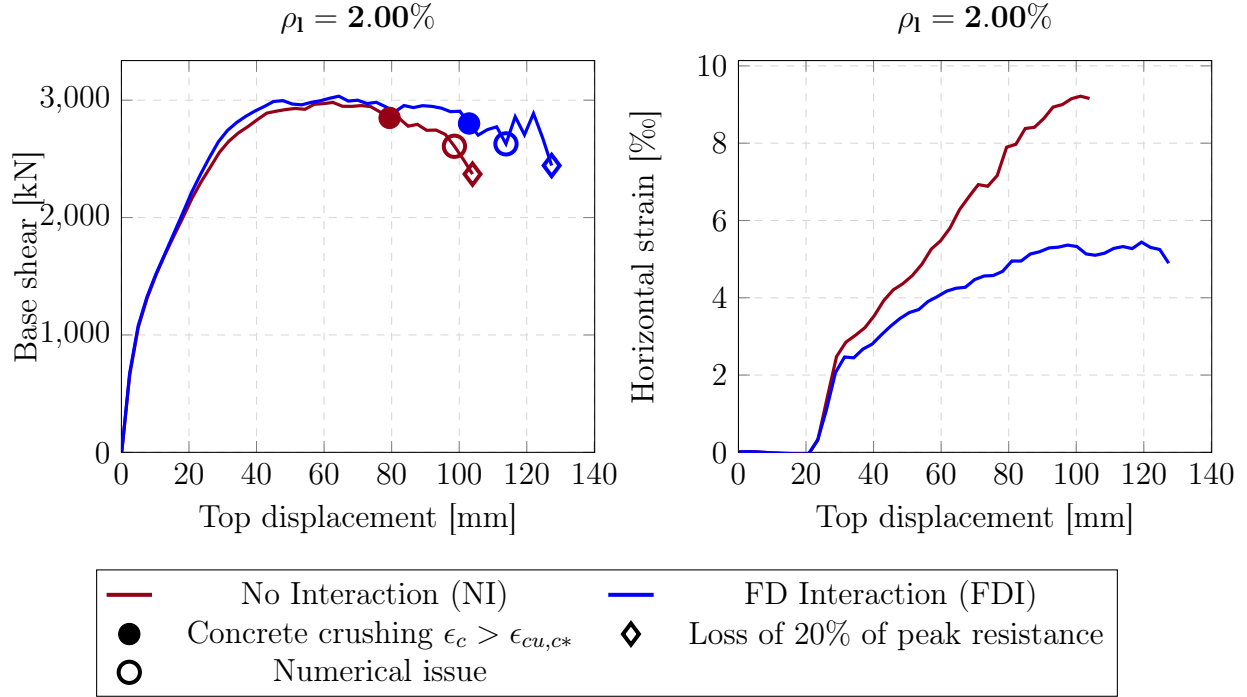


Figure 5.14: Force-displacement curves and horizontal strain of element np. 70 for  $\rho_l = 2.00\%$

This model presents a ratio  $\rho_t/\rho_l$  of less than 1/10, a value way below the recommended one by the Eurocode. The slab elements of 3.5 m width (thickness in FE model) actually showed noticeable strain and cracking as shown on Figure 5.15, defeating the purpose of a restraining floor diaphragm. Width was then raised to 7 m to prevent all horizontal strain of the floor diaphragm.

As presented on Figure 5.14, both force-displacement curves are almost identical up to a top displacement of 80 mm where discrepancies appear after the apparent crushing of concrete in the NI model. This concrete crushing at the toe is observed for a top displacement of 79 and 102 mm for the NI and FDI models respectively, an increase of 30%. Lateral load resistance degradation is observed in both models for 104 and 117 mm top displacement for the NI and FDI simulations respectively, a difference of 23%. Horizontal strain in element no. 70 is also reduced by up to 52% by the floor diaphragm in the FDI model.

Comparing these two models seem to show that considering the floor diaphragm slightly improves the ductility of the wall by 8 to 20% depending on the considered failure criteria. As those values are once again quite close, and they are also quite sensitive, they must be taken cautiously. Horizontal strain is also heavily reduced in the bottom section of the wall web.

$\rho_l = 3.00\%$

This model presents a ratio  $\rho_t/\rho_l$  of around than 6%, a value way below the recommended

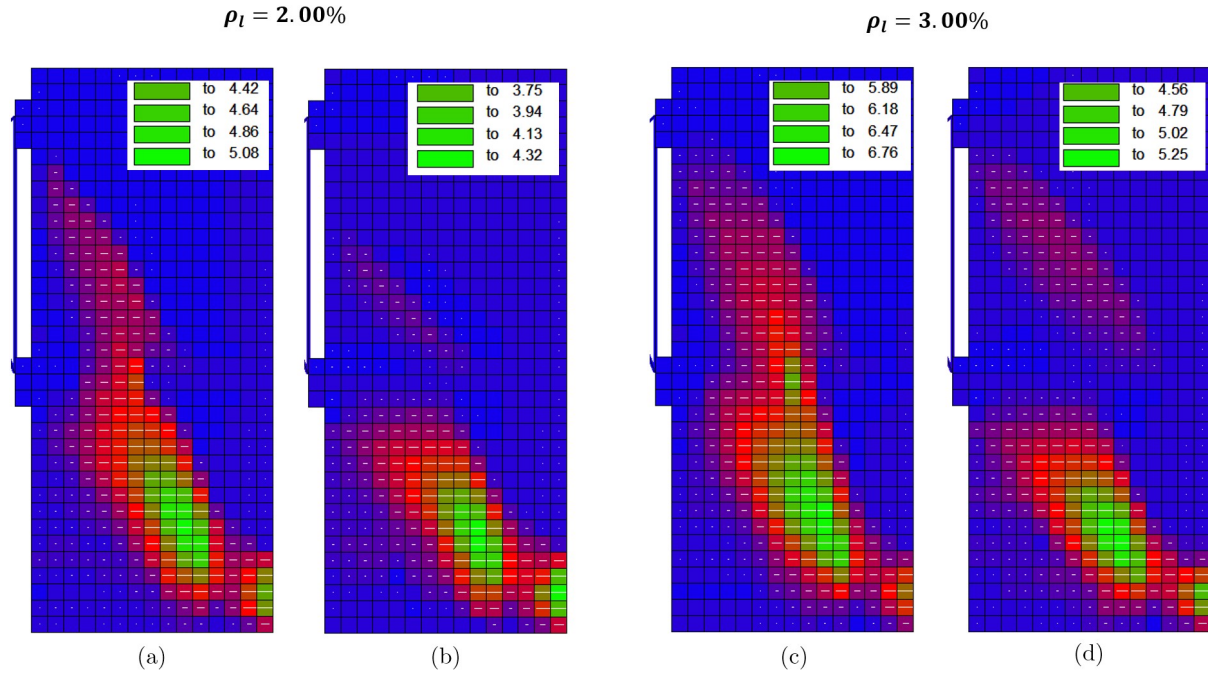


Figure 5.15: Difference of horizontal strain for different slab width (the FE thickness, out of the wall's plane): (a) and (c) : 3.5 m, (b) : 7 m, and (d) : 10 m.

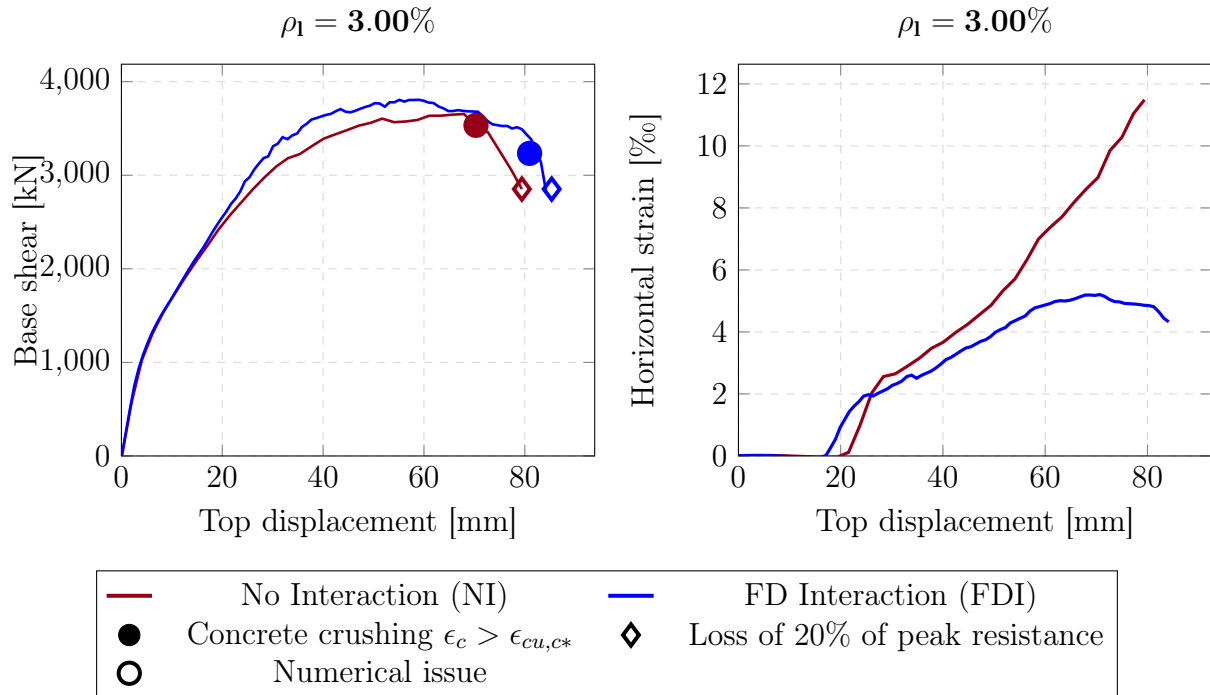


Figure 5.16: Force-displacement curves and horizontal strain of element np. 70 for  $\rho_l = 3.00\%$

one by the Eurocode which increases the shear vulnerability of the wall. Like for the previous model, the slab elements of 3.5 m width (thickness in FE model) actually showed noticeable strain and cracking as shown on Figure 5.15, defeating the purpose of a restraining floor diaphragm. Width was then raised to 7 m to prevent all horizontal strain

of the floor diaphragm.

Unlike most of the models presented beforehand, the two curves are here slightly different even before the peak resistance is reached. This is not caused by the increase of the floor slab dimensions. The FDI model presents a slightly higher peak resistance of 3706 kN while 3654 kN were observed in the NI model. Both models showed concrete crushing at the toe for top displacements of 70 and 81 mm for the NI and FDI simulations respectively, a 15% increase. In both cases, the concrete crushing then caused strain localization in those same elements.

Finally, resistance degradation was observed at top displacement of 79 mm for the NI model and 85 mm for the FDI model, coming after the localization those values are somewhat less reliable. As can be seen on Figure 5.16, horizontal strain is reduced by the presence of the floor diaphragm and for element no. 70, this difference can rise up to more than 59% towards the end of both curves.

The displacement capacity of the wall seems to be slightly affected by the floor diaphragm presence. This increase in ductility however is less obvious and important than for  $\rho_l = 2.00\%$ . Horizontal strain is also heavily reduced in the bottom section of the wall web.

|                              | $\rho_l = 0.50\%$ | $\rho_t = 1.00\%$ | $\rho_t = 2.00\%$ | $\rho_t = 3.00\%$ |
|------------------------------|-------------------|-------------------|-------------------|-------------------|
| Peak resistance              | +0%               | +0%               | +1%               | +4%               |
| <b>Displacement capacity</b> |                   |                   |                   |                   |
| - Concrete crushing          | +18%              | -9%               | +30%              | +15%              |
| - Resistance degradation     | +18%              | -10%              | +7%               | +16%              |

Table 5.4: Changes in resistance and displacement capacity for different longitudinal reinforcement ratios.

Increased vertical reinforcement is translated by a higher resistance, but also a lower displacement capacity since more concrete is needed to balance the tension forces in the steel. For higher values of lateral force and the same shear resistance, it appears that the floor diaphragm plays a more important role. For ratios  $\rho_t/\rho_{ho_l} < 10\%$ , the observed gain in ductility is of at least 13%. This value being relatively small and sensitive to multiple modelling parameters, it is to be considered carefully.

### 5.2.4 Aspect ratio

Different values of the aspect ratio were also analysed. By changing the length of the wall, while the shear span is unchanged, the aspect ratio is modified. The main focus of this thesis was first mainly on walls with rather aspect ratio ( $> 1.8$ ) but from the previous results, the impact of the floor diaphragm does not seem to really affect such walls. Some lower aspect ratios walls will therefore be studied.

**AR = 2.5**

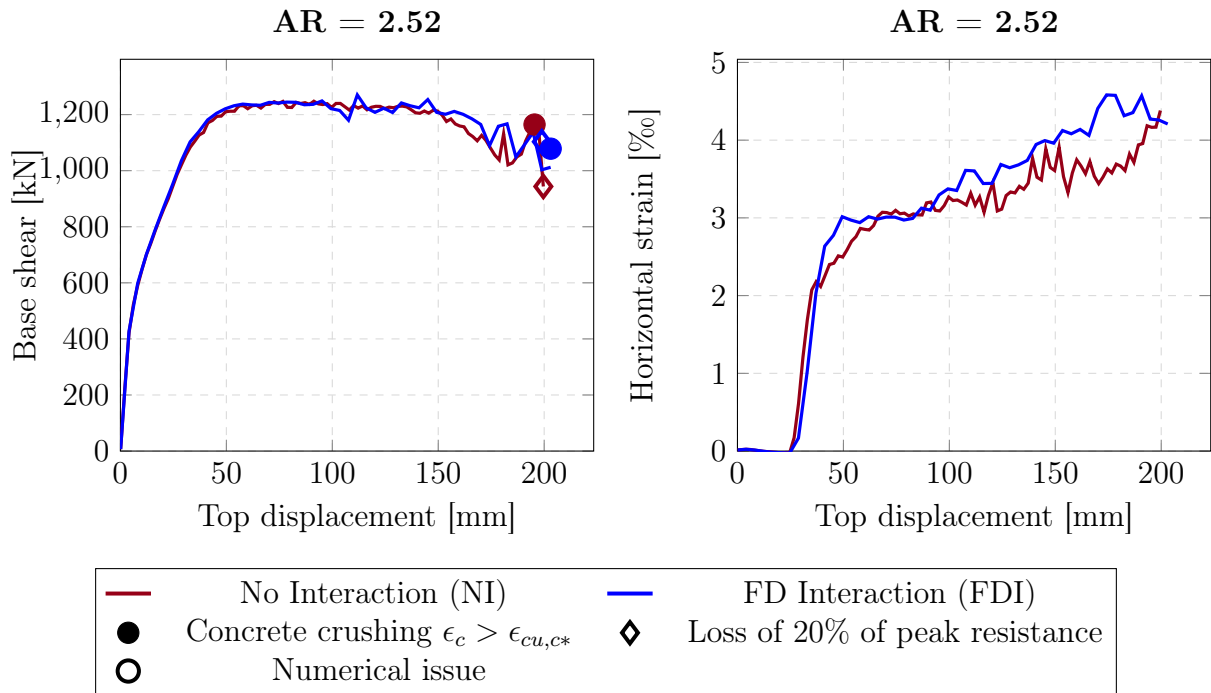


Figure 5.17: Force-displacement curves and horizontal strain of element np. 70 for AR of 2.52

Force displacement curves displayed on Figure 5.17 appear to be exactly the same. Concrete crushing at the toe is observed for a top displacement of 195 and 203 mm for the NI and FDI models respectively, an increase of 4%. There was no evidence of reinforcements rupture in the simulations. Lateral load resistance degradation is reached in both models for really similar top displacements of 199 and 198 mm top displacement for the NI and FDI simulations respectively. Abrupt change in resistance for the NI model occurred at the top displacement of 199 mm, discarding all the next load steps. There does not seem to be any significant horizontal strain reduction due to the presence of the floor diaphragm.

As expected, for a higher aspect ratio, shear plays a smaller role in the behaviour of the wall, it is therefore expected from the floor diaphragm to have a smaller influence as well. Cracks and main horizontal strain of the web did not reach the first floor, thus making it difficult for the diaphragm to come into play.

**AR = 1.45**

This model actually used 16 elements along the length of the wall instead of 15 for the previous ones. Force displacement curves displayed on Figure 5.18 are do not seem to significantly differ at any point. The combined shear-flexure mechanism is common to both simulation. Concrete crushing at the toe is observed for a top displacement of 104 and 111 mm for the NI and FDI models respectively, an increase of 6%. There was no evidence of reinforcements rupture in the simulations. Lateral load resistance degradation

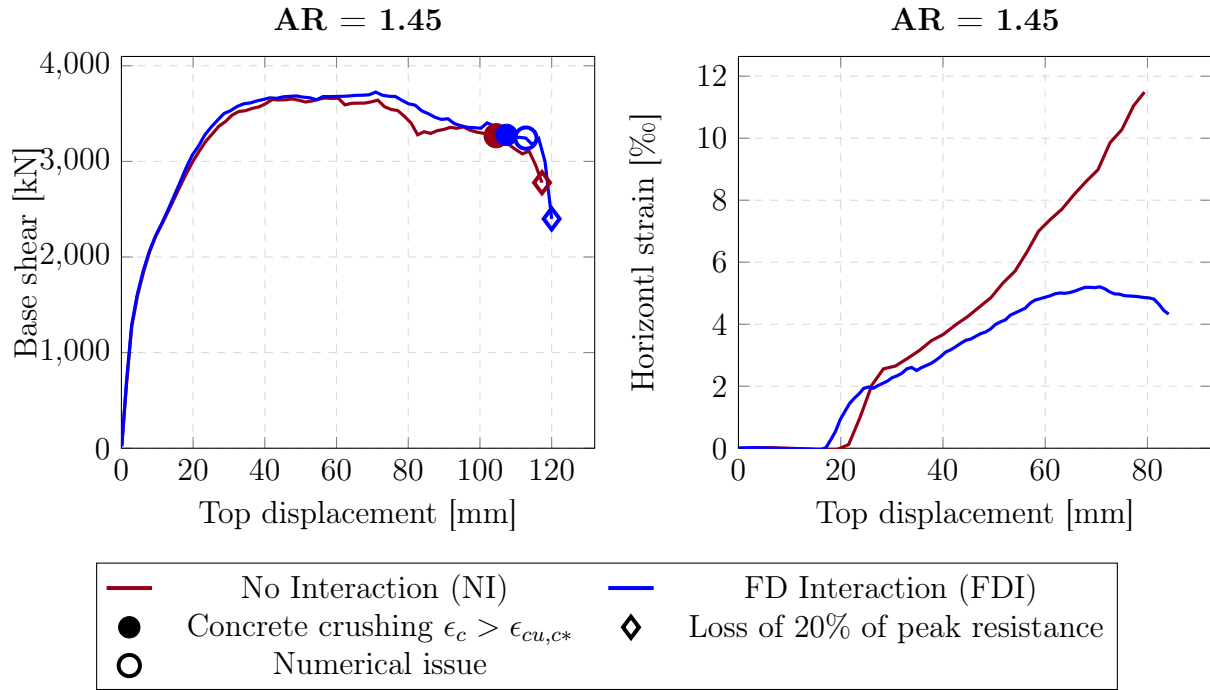


Figure 5.18: Force-displacement curves and horizontal strain of element np. 70 for AR of 1.45

is reached in both models for 117 and 123 mm top displacement for the NI and FDI simulations respectively, a difference of 5%.

Strain localization occurred at the same time as concrete crushing for the NI model. For the FDI simulation, strain localization occurred a little later at 120 mm top displacement, a difference of 15%. As can be seen on Figure 5.18, horizontal strain for the element no. 78 are reduced by the presence of the floor diaphragm, this difference can rise up to more than 50%.

While reduced aspect ratio is expected to exacerbate the shear vulnerability of the wall and therefore increase the influence of the floor diaphragm, it does not appear to be the case in the results. Only very small increase in displacement capacity was observed. This could be related to the fact that the mesh is relatively more coarse given the augmented length of the wall.

### AR = 1.2

This aspect ratio is achieved by increasing the wall length to 4.6 m over which the number of finite elements is actually 16 instead of 15. For reasons explained in Section 5.2.3 (cf. Figure 5.15), the width of the floor slab is augmented to 10 m. The force-displacement curves shown on Figure 5.19 are highly similar until the concrete crushing in the NI model occurs, after which the curves follow different paths. This concrete crushing at the toe is observed for a top displacement of 60 and 84 mm for the NI and FDI models respectively, a significant increase of nearly 40%. None of the simulations showed rupture

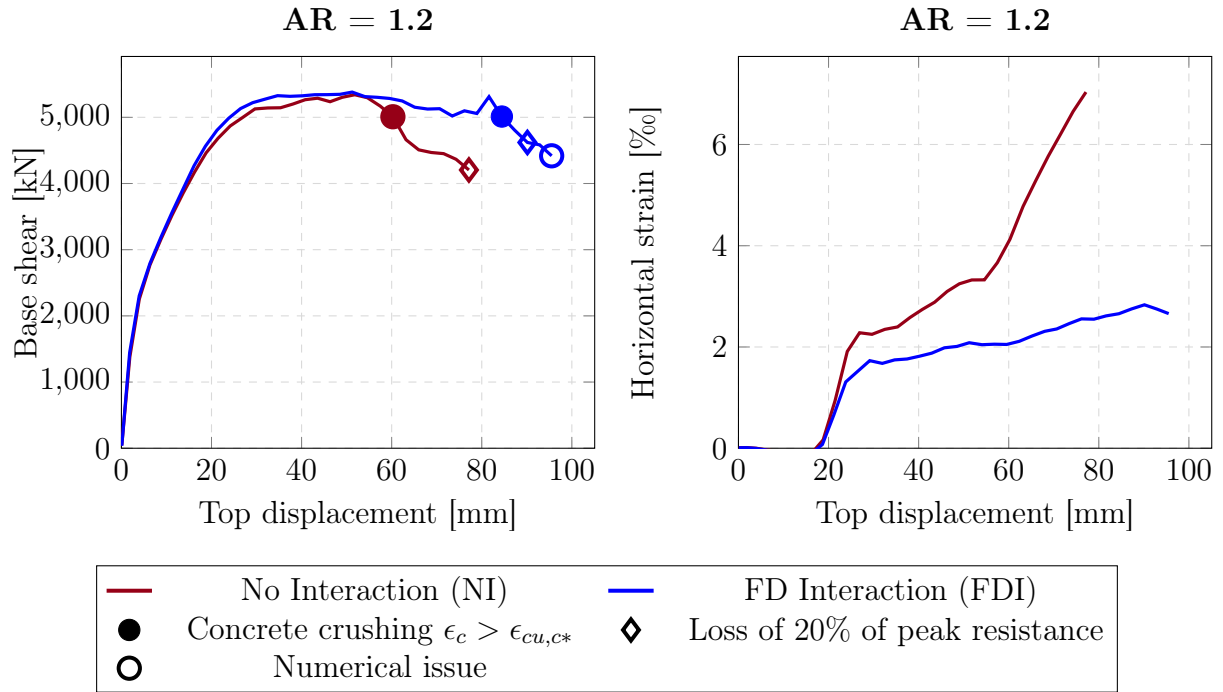


Figure 5.19: Force-displacement curves and horizontal strain of element np. 70 for AR of 1.2

of reinforcements in tension. Both models suffered lateral load resistance degradation. The NI simulations lost 20% of its peak resistance for a top displacement of 77 mm while the FDI showed similar degradation at 90 mm top displacement. Strain localization however occurred before the strength degradation criteria was met for the NI model, which make it less reliable. For the FDI model, this localization took place for a 95 mm top displacement. Horizontal strain of element 60 displayed in Figure 5.19 shows how the floor diaphragm reduces the magnitude of strain in the bottom part of the wall. The reduction from the simpler model can be as high as 63% for this precise element.

The force-displacement curve clearly shows an increase of ductility for this wall configuration. Concrete crushing at the toe was delayed by more than 40% with the floor FD Interaction (FDI). The mesh dimensions which could be considered quite coarse may be at the origin of this extra ductility, but as both NI and FDI models used the same mesh. When compared to the walls with different aspect ratios, the increase seems somewhat excessive.

### AR = 0.9

This low aspect ratio is obtained by increasing the length of the wall to 6 m. More elements are needed along this length to prevent the mesh from being too coarse. 20 elements instead of 15 are therefore used for this model. For reasons explained in Section 5.2.3 (cf. Figure 5.15), the width of the floor slab is augmented to 15 m. Unlike the previous models, a slight over-strength is observed when comparing the FDI simulation with the NI model. Peak resistance goes from 8233 kN to 8559 kN, an increase of 4%. The difference of resistance is quite small but the force-displacement curves show more

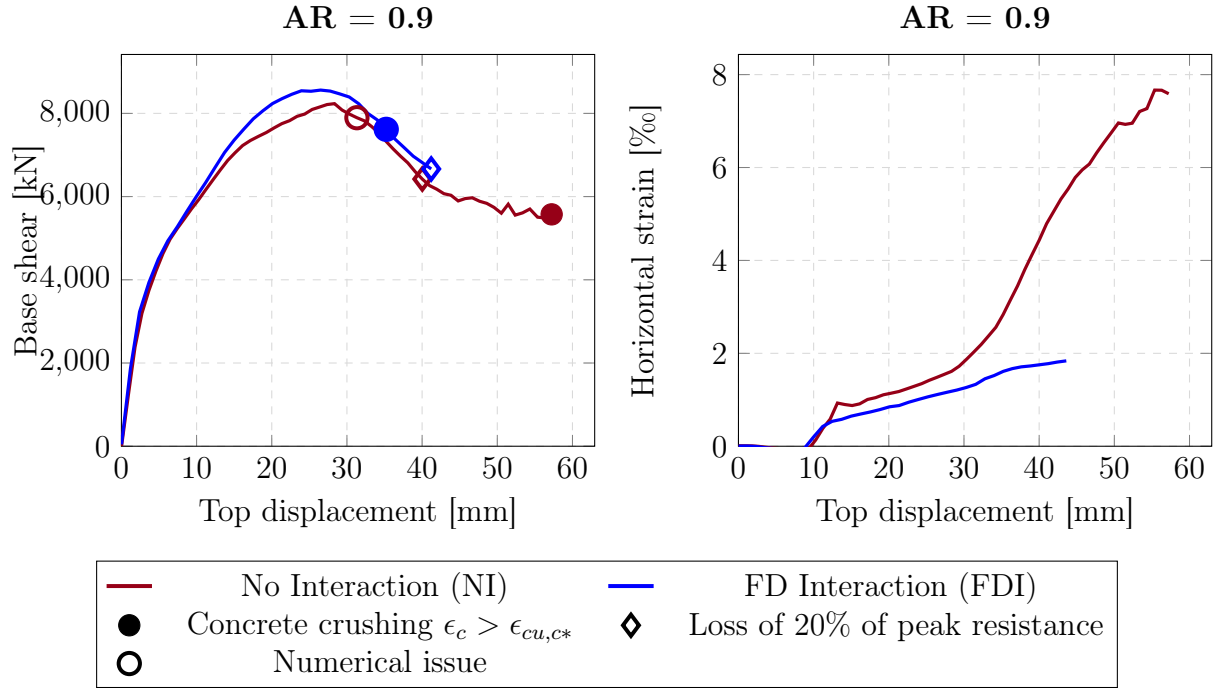


Figure 5.20: Force-displacement curves and horizontal strain of element np. 70 for AR of 0.9

disparities than the previous models, the energy dissipated by the FDI model up to the peak being slightly larger than for the NI model.

Crushing of concrete in the NI model occurs surprisingly late at a top displacement of 57 mm while concrete crushing is observed for the FDI model at 35 mm top displacement only. Strain localization occurs in both model at 31 at 35 mm top displacement for the NI and FDI models respectively. For the simulation considering the floor diaphragm, this localization occurs simultaneously with the concrete crushing. The following results, such as the resistance degradation for 40 and 41 mm top displacement, are thus not reliable. horizontal strain is effectively reduced by the floor diaphragm as seen on Figure 5.20 with the difference being as has 37%.

This very squat wall has exhibited a higher peak resistance when the floor diaphragm was considered. It seems that the latter has restrained the lateral load resistance degradation due to the shear deformations. However the ductility does not seem to be increased which is surprising as the opposite case was actually expected. However, the model being less reliable for very low aspect ratio walls and the mesh being quite coarse, those results and observations are to be taken cautiously.

For high aspect ratio, the floor diaphragm did not play any role regarding the behaviour of the wall or the horizontal strain reduction. For lower aspect ratios, the results are varied. While it was expected that squatter walls would be more shear critical and therefore the influence of the stirrup effect would be greater, only the aspect ratio of 1.2 exhibited such a phenomenon. The models used for this analysis seem to be less adapted to these lower aspect ratio walls, given the variability of the results.

|                                 | AR = 2.52 | AR = 1.52 | AR = 1.2 | AR = 0.9 |
|---------------------------------|-----------|-----------|----------|----------|
| Peak resistance                 | +2%       | +2%       | +1%      | +4%      |
| <b>Displacement capacity</b>    |           |           |          |          |
| - <i>Concrete crushing</i>      | +4%       | +3%       | +40%     | -38%     |
| - <i>Resistance degradation</i> | +0%       | +2%       | +17%     | +3%      |

Table 5.5: Changes in resistance and displacement capacity for different aspect ratios

## 5.3 Targeted analysis

The previously detailed study did not showcase an obvious impact of the floor diaphragm interaction on the shear wall through the sought after "stirrup effect". Other simulations combining different values of the parameters were then run.

### 5.3.1 Reinforcements ratio $\rho_t/\rho_l$

The previous analysis suggests that the ratio between the transverse and longitudinal reinforcements quantities is a defining factor of the stirrup effect. The lowest ratio of the previous analysis was 6%, and it was the case with the most noticeable difference between the NI and FDI cases. Reducing further the proportion of transverse reinforcement compared with the longitudinal reinforcements is therefore expected to better highlight a possible effect of floor diaphragm as stirrup. Such high difference between longitudinal and transverse reinforcement may however seem unrealistic for recently designed walls, as the Eurocode recommends  $\rho_t > 0.25\rho_l$ .

$\rho_l = 3\% ; \rho_t = 0.1\% ; \text{ALR} = 25\%$

The first wall is then fitted with quite high longitudinal reinforcements quantity  $\rho_l = 3.00\%$  and very low transverse reinforcements ratio  $\rho_t = 0.10\%$ . This gives a ratio  $\rho_t/\rho_l$  of around 3.3%.

The force-displacement curves shown in Figure 5.21 present more evident differences when compared to the previous models. Peak resistance of the FDI model reaches 4165 kN, a nearly 12% increase from the 3729 kN of the NI model for the same top displacement of around 45 mm. This noticeable increase is of course due to the floor diaphragm restraint on the wall. The NI wall fails due to brittle combined shear-flexure but exhibits very important diagonal cracking and its behaviour is greatly influenced by this shear deformation. The floor diaphragm prevents the expansion of the section and limiting the shear deformation of the wall as the Figure 5.23 shows. This allows the longitudinal reinforcements in tension to deform more for an identical top displacement, dissipating more energy. As their tensile strain are larger, the total bending moment and lateral load resistance are also higher.

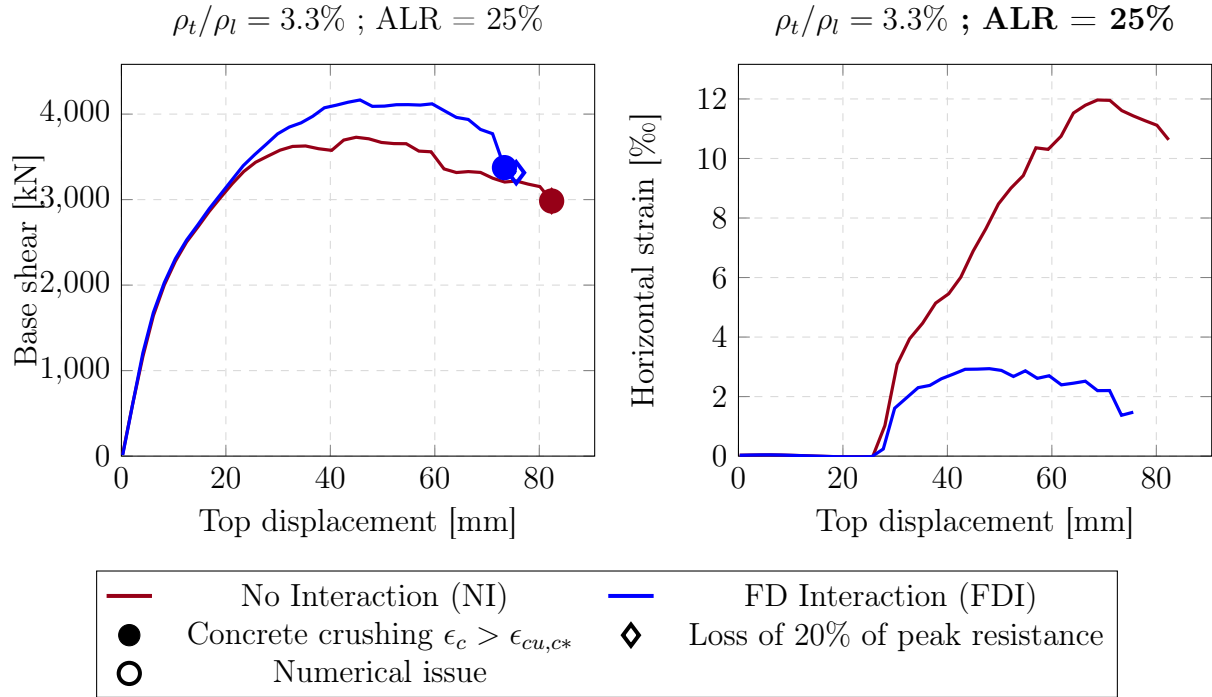


Figure 5.21: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 3\%$  ;  $\rho_t = 0.1\%$  ;  $\text{ALR} = 25\%$

Displacement capacity however does not seem to be increased. A small decrease is even observed concrete crushing takes place for a top displacement of 73 mm for the FDI model whereas for the NI model it was 82 mm, a difference of -11%. Strain localization both occur at those same time steps. Finally, lateral load resistance degradation is reached for the NI model at 82 mm and for the FDI model at 75 mm, again a difference of around 9%. Horizontal strain is drastically reduced by as much as 78% for element no. 70 as seen in Figure 5.21.

Increasing the lateral load resistance but reducing the displacement capacity causes the area below the curve to actually be only 1% larger in the FDI case.

This model therefore shows that for a very low shear reinforcement ratio compared to the longitudinal reinforcement quantity ( $\rho_t/\rho_l = 3.3\%$  here), the floor diaphragm can actually impact the behaviour of the shear wall. The resistance is increased by almost 12% and the displacement capacity is on another hand slightly reduced by 11%. As it seems that this ratio between transverse and longitudinal reinforcement has the most impact on the stirrup effect amplitude, different values of this ratio are then tested to confirm the supposed trend.

$\rho_l = 3.00\% ; \rho_t = 0.15\% ; \text{ALR} = 25\%$

The transverse ratio is then increased to 0.15% and thus  $\rho_t/\rho_l = 5\%$ .

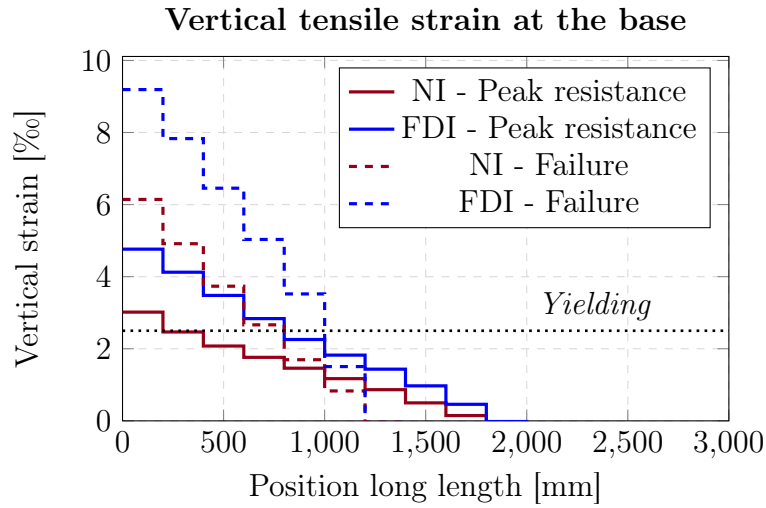


Figure 5.22: Vertical strain at the base of the wall for peak resistance and failure showing the difference in longitudinal rebars energy dissipation

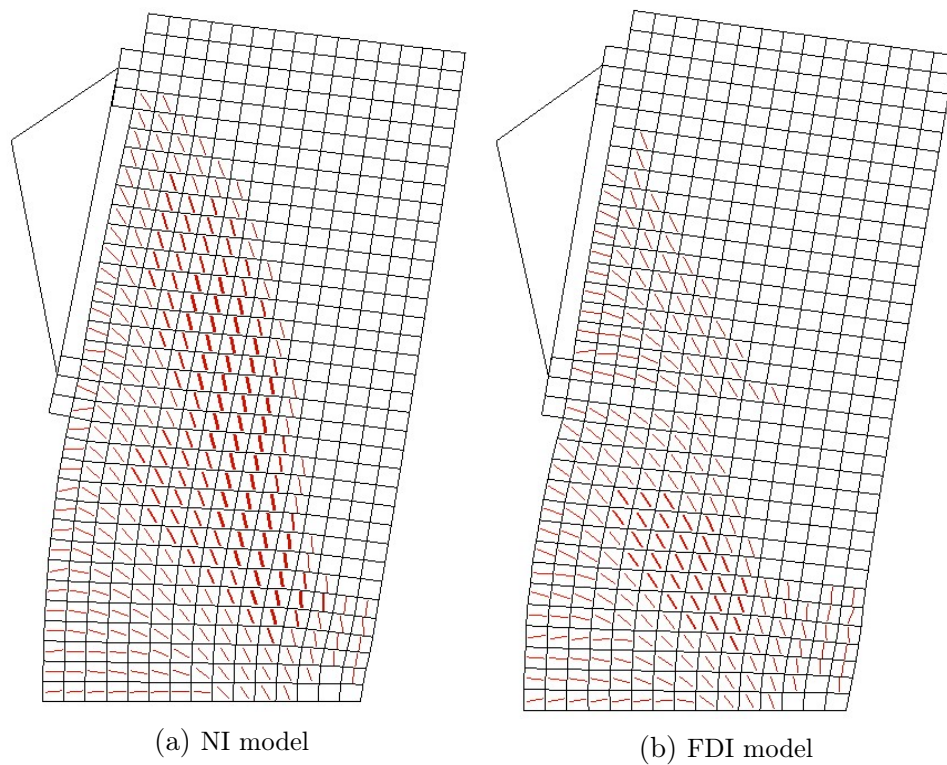


Figure 5.23: Deformation and crack pattern of both models at their respective peak load.

For this value of transverse reinforcement, the FDI model exhibits a higher peak resistance as well as at 4224, a 9% gain from the 3880 kN of the NI model. As Figure reffigCurv-M-VR3-TR15 shows, this peak occurs for a slightly higher top displacement of 57 mm compared to the 49 mm of the NI model. The concrete crushing occurs for a top displacement of around 72 mm for both simulation. Strain localization is also observed right at the same step for the FDI model and lightly later for the NI case. Lateral load resistance

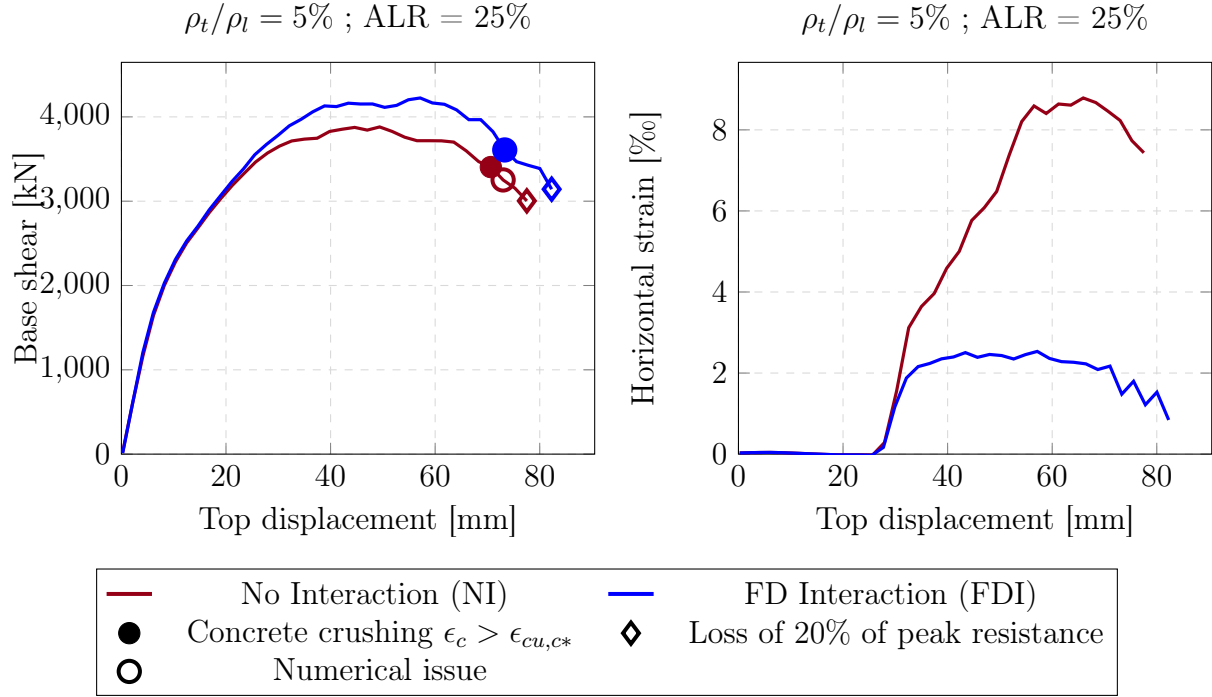


Figure 5.24: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 3\%$  ;  $\rho_t = 0.15\%$  ;  $\text{ALR} = 25\%$

degradation of -20% is reached for both models for top displacements of 77 and mm respectively. Horizontal strain in element no. 70 is reduced by as much as 74%.

The peak resistance sees a noticeable gain, while the displacement capacity is unchanged. This results in an augmentation of the energy dissipation capacity regarding the area below the force-displacement curve of around 11%.

$$\rho_l = 3.00\%; \rho_t = 0.20\% ; \text{ALR} = 25\%$$

Increasing the transverse reinforcement ratio even more to 0.20% ( $\rho_t/\rho_l = 7\%$ ) results in an increase of the peak lateral load resistance by 7% going from 4287 to 4019 kN. This peak value is reached at a higher top displacement of 55 mm in the FDI than in the NI simulation where it was reached at 47 mm. Concrete crushing is observed at a later stage (76 mm top displacement) in the FDI simulation than in the NI model (68 mm top displacement). Lateral resistance degradation of -20% is detected for similar top displacement of 84 and 80 mm for the NI and FDI models respectively. Reduction of p to 60% of the strain in element no. 70 is observed when compared the two models.

The increase in peak resistance of 7% is noticeable, but is less important than for the lower values of  $\rho_t$ . Displacement capacity does not seem to be affected by the presence of the floor diaphragm in this configuration. The increase of area below the curve amounts to 7% as well. par

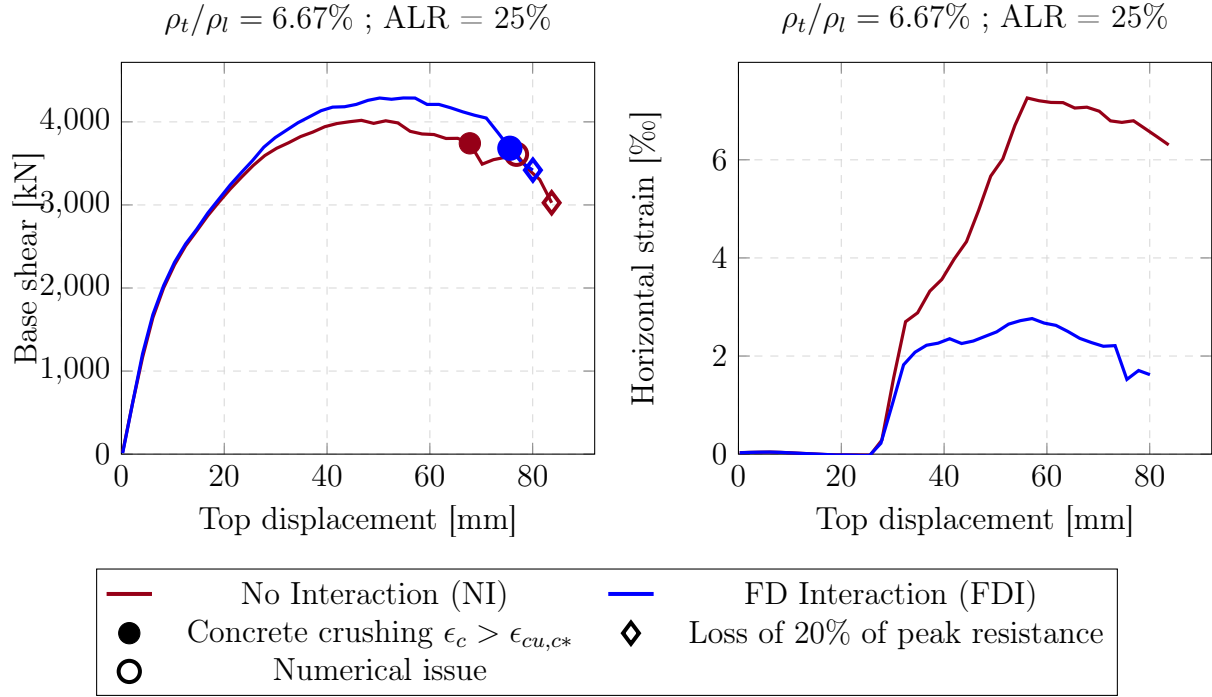


Figure 5.25: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 3\% ; \rho_t = 0.20\% ; \text{ALR} = 25\%$

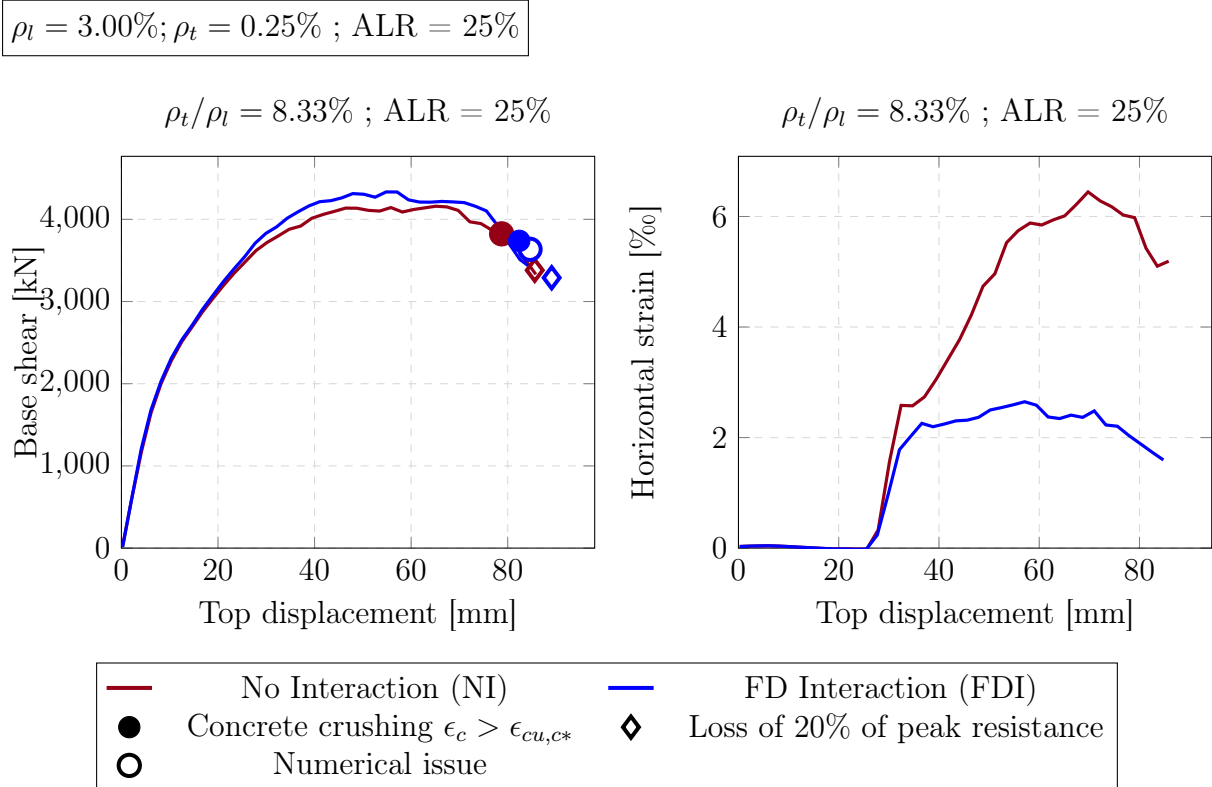


Figure 5.26: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 3\% ; \rho_t = 0.25\% ; \text{ALR} = 25\%$

Further increasing the transverse reinforcement causes the gain of peak resistance to drop to around 4% as the FDI model exhibits a strength of 4333 kN whereas the NI showcased

resistance of 4159 kN. Displacement capacity is not significantly altered either and all the different failure criteria occur for top displacements of 80 to 85 mm. Horizontal strain is reduced by up to 64% when considering the floor diaphragm.

$$\rho_l = 3.00\%; \rho_t = 0.30\% ; \text{ALR} = 25\%$$

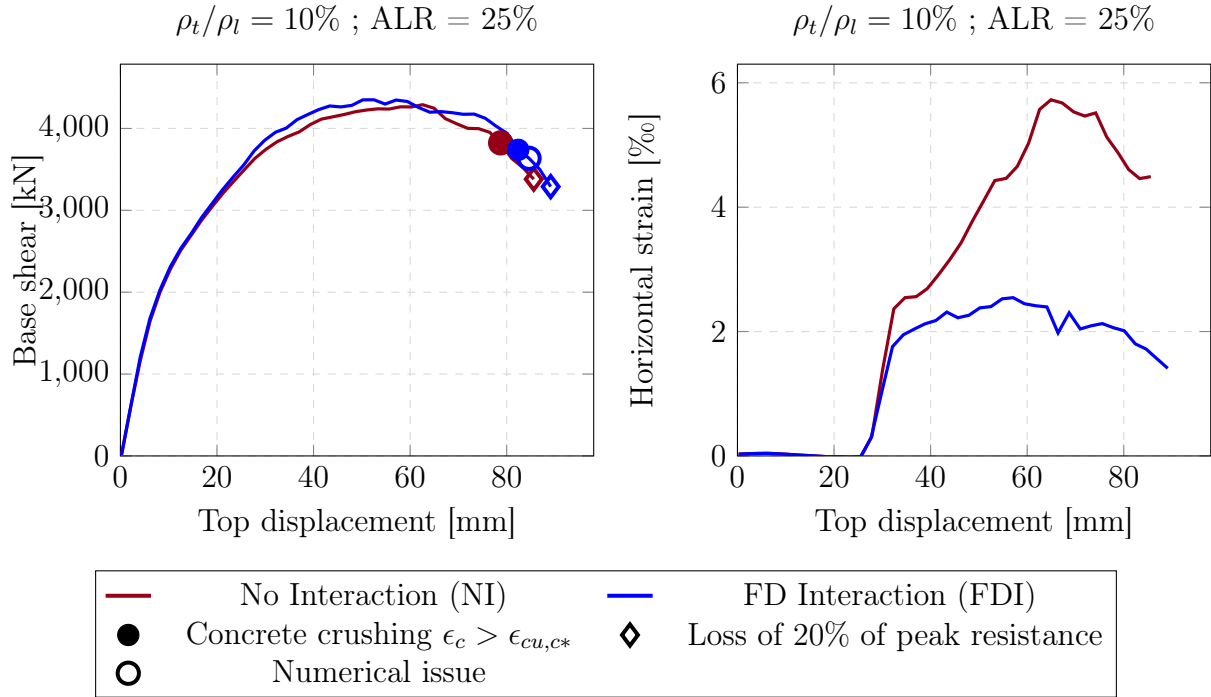


Figure 5.27: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 3\%$  ;  $\rho_t = 0.30\%$  ;  $\text{ALR} = 25\%$

For a ratio  $\rho_t/\rho_l$  of 10%, Figure 5.27 shows almost identical force-displacement curves. This is not a surprise as it was already the case in Figure 5.14. The peak resistance of 4349 kN the FDI is only 1% above the NI resistance of 4289 kN. The different failure criteria occur at very close load steps for both models around 80 mm top displacement. The displacement capacity of the wall is therefore not significantly modified.

Other simulations using a longitudinal reinforcement ratio  $\rho_l$  of 2.00% gave results similar to the ones previously described. Their force-displacement curves are presented in Appendix A

### 5.3.2 Low aspect ratio

The aspect ratio has also been defined as a possible factor which could highlight a possible stirrup effect. While the peak resistance did not seem affected in the parametric analysis, the 1.2 aspect ratio wall displayed significantly enhanced displacement capacity. It will then be modelled for different  $\rho_t/\rho_l$  ratios.

$$\rho_l = 2.00\%; \rho_t = 0.10\%; \text{ALR} = 25\%; \text{AR} = 1.2$$

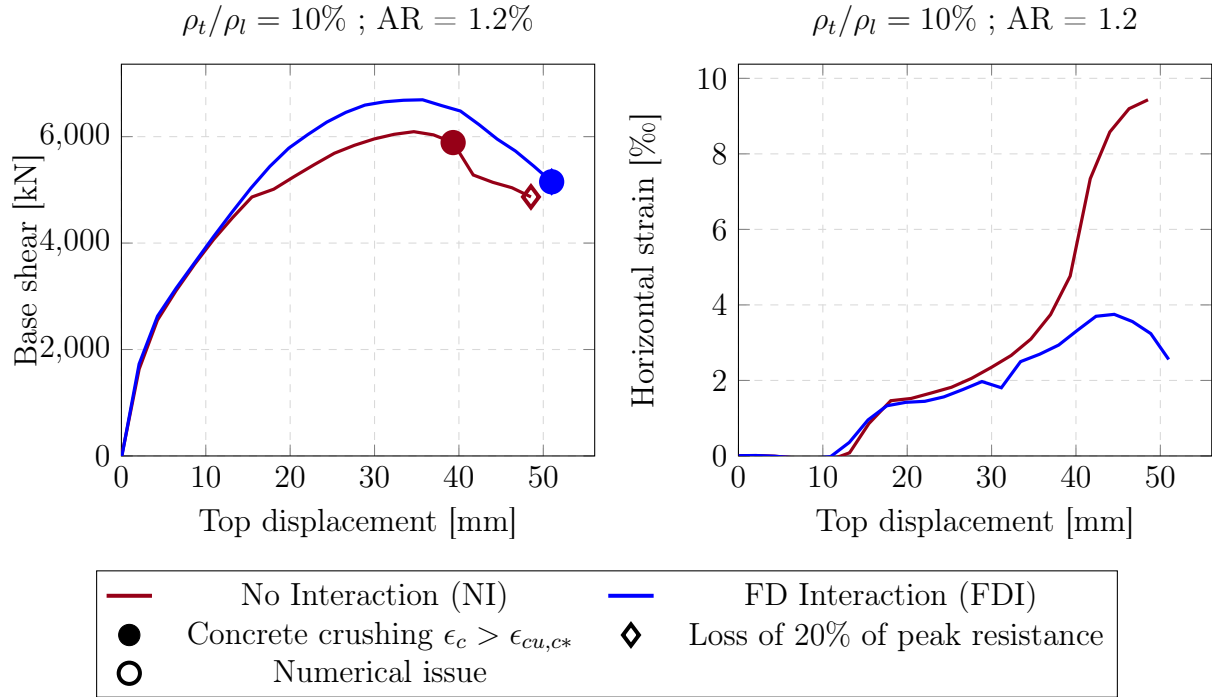


Figure 5.28: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 2\%$  ;  $\rho_t = 0.10\%$  ;  $\text{ALR} = 25\%$  ;  $\text{AR} = 1.2$

Peak resistance of 6692 kN for the FDI model is a 10% increase from the 6093 kN resistance of NI model. Concrete crushing is also delayed from 39 mm to 51 mm top displacement (+30%) whereas the resistance degradation is only increased by 5%. While shear deformations are restrained, as it can be seen on Figure 5.28 with element no. 70, the global failure mechanism of the wall is unchanged as showed by Figure 5.29.

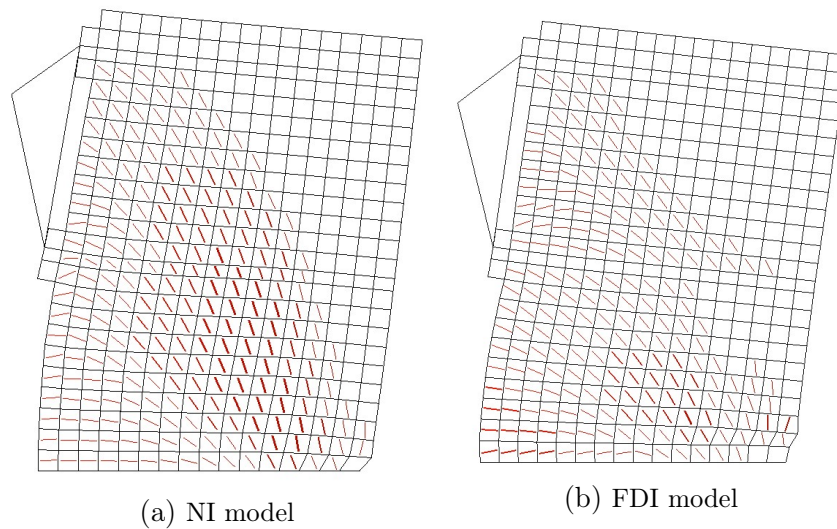


Figure 5.29: Deformation and crack pattern of both models at their respective peak load.

$$\rho_l = 2.00\%; \rho_t = 0.15\%; \text{ALR} = 25\%; \text{AR} = 1.2$$

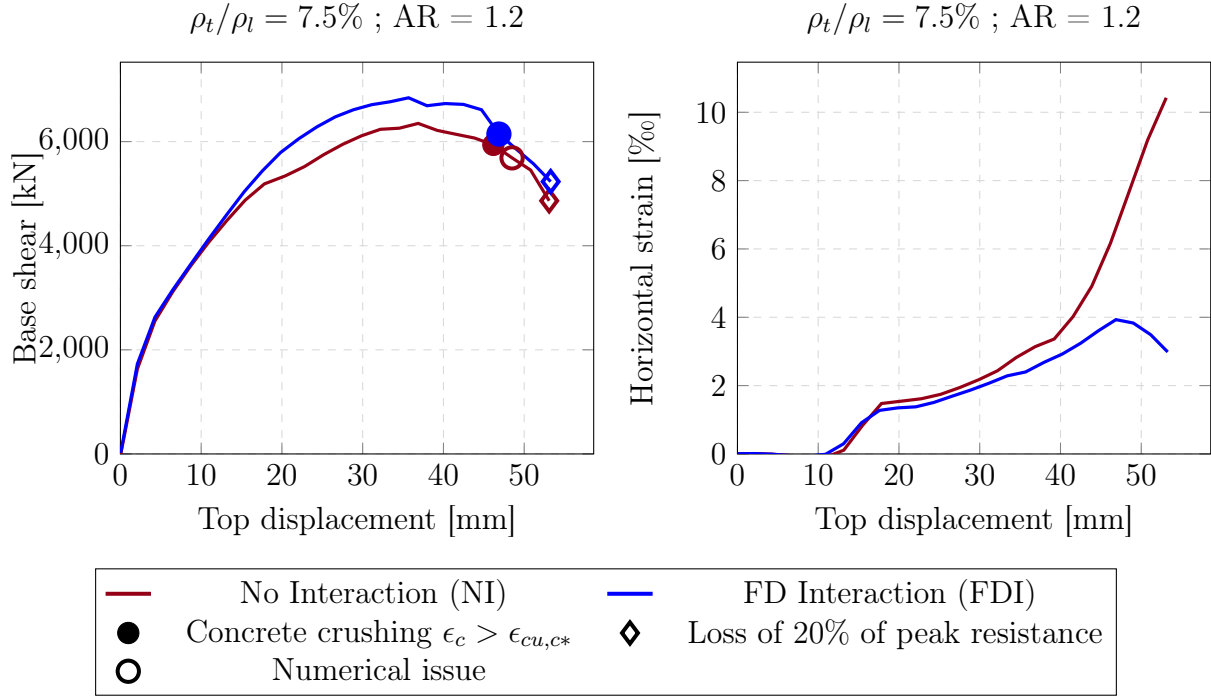


Figure 5.30: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 2\%$  ;  $\rho_t = 0.15\%$  ;  $\text{ALR} = 25\%$  ;  $\text{AR} = 1.2$

Resistance is increased by 8% in this configuration, going from 6349 kN to 6841 kN in the FDI model. Concrete crushing and resistance degradation criteria are met for similar top displacement of respectively 46 mm and 53 mm in both models. The displacement capacity therefore does not seem to be affected.

$$\rho_l = 2.00\%; \rho_t = 0.20\%; \text{ALR} = 25\%; \text{AR} = 1.2$$

Resistance of the FDI model is 6769 kN, a 7% gain from the 6346 kN of the NI model. Concrete crushing and resistance degradation criteria are once again met for similar top displacement of respectively 45 mm and 54 mm in both models. The displacement capacity therefore does not seem to be affected.

These three models did show a greater impact of the stirrup effect of the wall's resistance given a lower aspect ratio when comparing for the same reinforcement ratios and ALR. It highlighted once again the impact of the  $\rho_t/\rho_l$  ratio on the resistance increase.

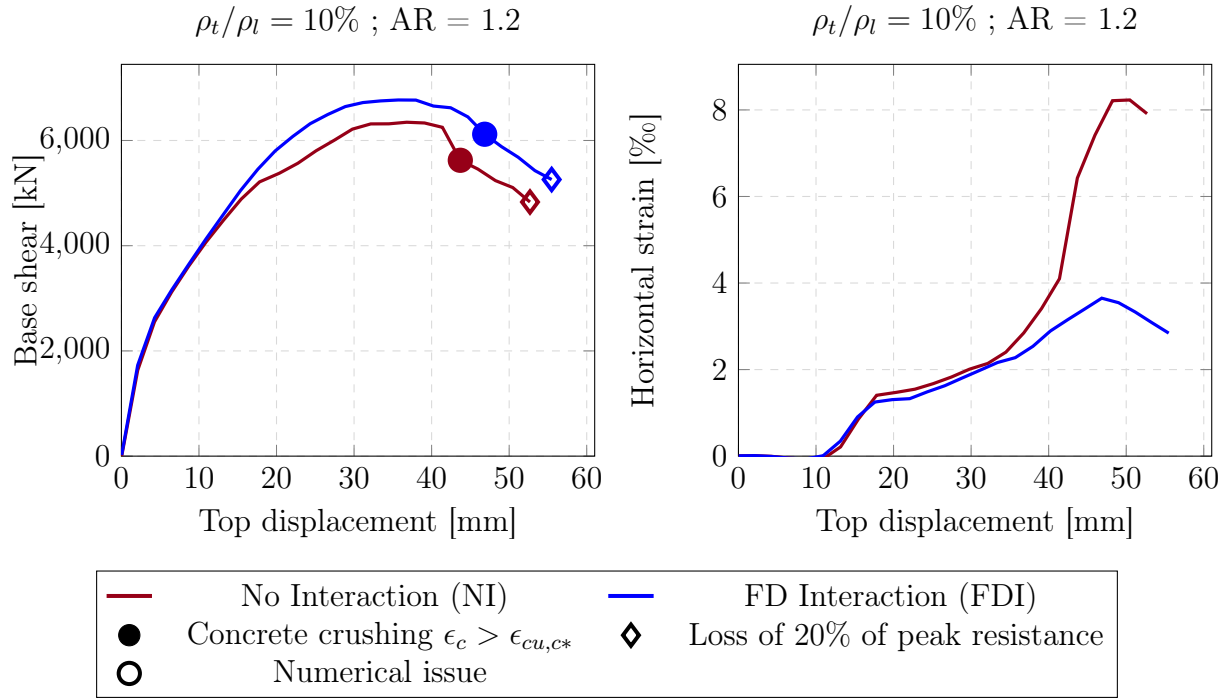


Figure 5.31: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 2\%$  ;  $\rho_t = 0.20\%$  ;  $ALR = 25\%$  ;  $AR = 1.2$

## 5.4 Takeaways

The first parametric analysis did not allow to clearly assess the impact of the floor diaphragm on the behaviour of the studied shear walls. However, the more targeted analysis has highlighted a possible influence of the stirrup effect on the behaviour of walls.

### 5.4.1 Displacement capacity

From the first parametric analysis, some configurations exhibited noticeable increase in displacement capacity (Figures 5.4, 5.14, 5.19) up to 40%. The ore targeted analysis also showcased a few configurations with varying ductility. However, the absence of real trend for this increase as well as the relatively small differences for the rest of the models do not allow to clearly establish a link between the floor diaphragm presence and a possible increase or reduction of ductility for the observed parameters.

The subjective character of the selected failure criteria, namely the concrete crushing and the resistance degradation, also interferes with a clear assessment given the small differences observed for both those criteria.

### 5.4.2 Lateral load resistance

From the initial parametric analysis, no impact on the resistance of the walls was noticeably detected. Apart from two models (see Figures 5.16 and 5.20) no difference larger than 1% was observed regarding the base shear resistance.

On another hand, the more targeted analysis allowed to highlight the influence of the reinforcement ratios on the stirrup effect. More precisely, it is the ratio between transverse and longitudinal reinforcement  $\rho_t/\rho_l$  which is the parameter most correlated with the magnitude of the stirrup effect.

The different models previously described have demonstrated that by reducing the shear deformation, the wall is able to dissipate more energy for a same displacement. This result in an increase of resistance which can be as high as 12% in some case. This effect was only perceived for walls fitted with very low  $\rho_t/\rho_l$  ratios of below 10%. This observation is presented in Figure 5.32. It is worth reminding that the slab width used in those models was 10 m which may be an unrealistic value but was used to highlight the stirrup effect possible influence in n extreme case. Using a more realistic value of effective width may therefore reduce the observed increase.

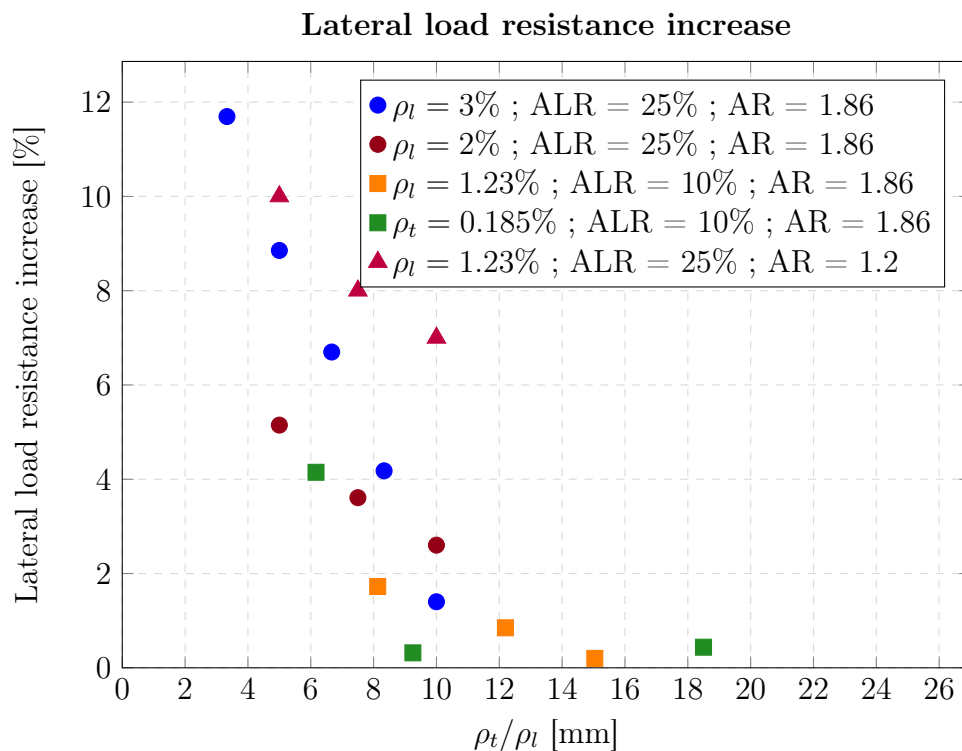


Figure 5.32: Increase in lateral load resistance of the wall due to the stirrup effect measured for different  $\rho_t/\rho_l$  ratios



## Chapter 6

# Slab coupling

Considering shear walls as being linked by coupling beams is fairly common and used in practice. On the contrary, the slab coupling of shear walls is rarely considered given the weaker bending stiffness of floor diaphragms. However, observations from [37], [38] and [17] reported that the slab coupling of shear walls could have an impact similar to the st. In this context, a parametric analysis was conducted to verify the extent of this coupling mechanism through floor slabs.

The bending stiffness of the slab being the main parameter behind the effectiveness of the coupling, different variables affecting this stiffness will be tested. The spacing between the two walls and the effective width of the slab will be investigated. The axial load ratio of the walls will also be considered as it increases the bending resistance of walls.

### 6.1 Base model

The wall configuration on which this analysis is based is derived from [17]. The four tested walls are simplified into one single frame of two walls and three slabs. The dimensions and reinforcements of the walls are kept exactly as described in Figure 4.12 and are listed in Figure 6.1.

The threshold value for the steel reinforcements ultimate strain is taken as  $0.6 \cdot \epsilon_{su} = 9\%$  as for the "stirrup effect" analysis.

The slab width is defined as 2 m, representing the distance between parallel walls which was actually 1.9 m in the tested setup.

The wall is modelled using fifteen 50 by 50 mm elements along its length and the slabs are modelled using 8 elements along their thickness as well as discrete reinforcements as described in Section 4.4. The load distribution is also the same linear distribution as for the validation model, with the first floor representing 17% of the total base shear and the second and third floor respectively accounting for 33% and 50%.

|                                |                    |      |     |
|--------------------------------|--------------------|------|-----|
| <b>Wall dimensions</b>         |                    |      |     |
| First floor height             | $h_1$              | 1.5  | m   |
| Second floor height            | $h_2$              | 3    | m   |
| Second floor height            | $h_2$              | 4.5  | m   |
| Length                         | $l_w$              | 750  | mm  |
| Thickness                      | $t$                | 350  | mm  |
| Aspect ratio                   | $L_s/l_w$          | 4.62 | /   |
| <b>Concrete</b>                |                    |      |     |
| Compression strength           | $f'_c$             | 27   | MPa |
| Ultimate strain                | $\epsilon_{cu,c*}$ | 1.5  | %   |
| <b>Reinforcements</b>          |                    |      |     |
| Vertical reinforcement ratio   | $\rho_l$           | 0.8  | %   |
| Transverse reinforcement ratio | $\rho_t$           | 0.75 | %   |
| Ultimate strain                | $\epsilon_{su}$    | 15   | %   |
| Axial load ratio               | $N/A_c f_c$        | 2    | %   |
| <b>Floor diaphragm</b>         |                    |      |     |
| Thickness                      | $t_{fd}$           | 80   | mm  |
| Width                          | $Y$                | 2    | m   |

Table 6.1: Reference wall properties

## 6.2 Full slab width

The slab length defines the distance between the two supposedly coupled walls. Increasing this distance reduces the rotational and transversal stiffnesses of the slab which are given by :

$$\frac{M}{\theta} = \frac{6EI}{L^3} (L + W))^2 \quad (6.1)$$

$$\frac{V}{\delta} = \frac{12EI}{L^3} \quad (6.2)$$

At first, the effective width of the slab is taken as the actual width of the slab being 2 m. In the test described in Section 4.4 the whole slab experienced deformations and cracking, meaning that all of the slab was somewhat activated and contributed to the coupling effect. As Vector2 is a 2D FE software, it was not possible to conduct an analysis on the real effective width of the slab.

|                      | $L=0.5$ m | $L=0.75$ m | $L=1$ m | $L=2$ m |
|----------------------|-----------|------------|---------|---------|
| Peak resistance [kN] | 98.5      | 90.4       | 86.2    | 74.2    |
| Increase             | 87%       | 71%        | 63%     | 41%     |

Table 6.2: Peak resistances for different slab length

The force-displacement curves of Figure 6.1 and the peak resistance values of Table 6.5 highlight the compelling impact of the coupling slabs. The lateral load resistance of

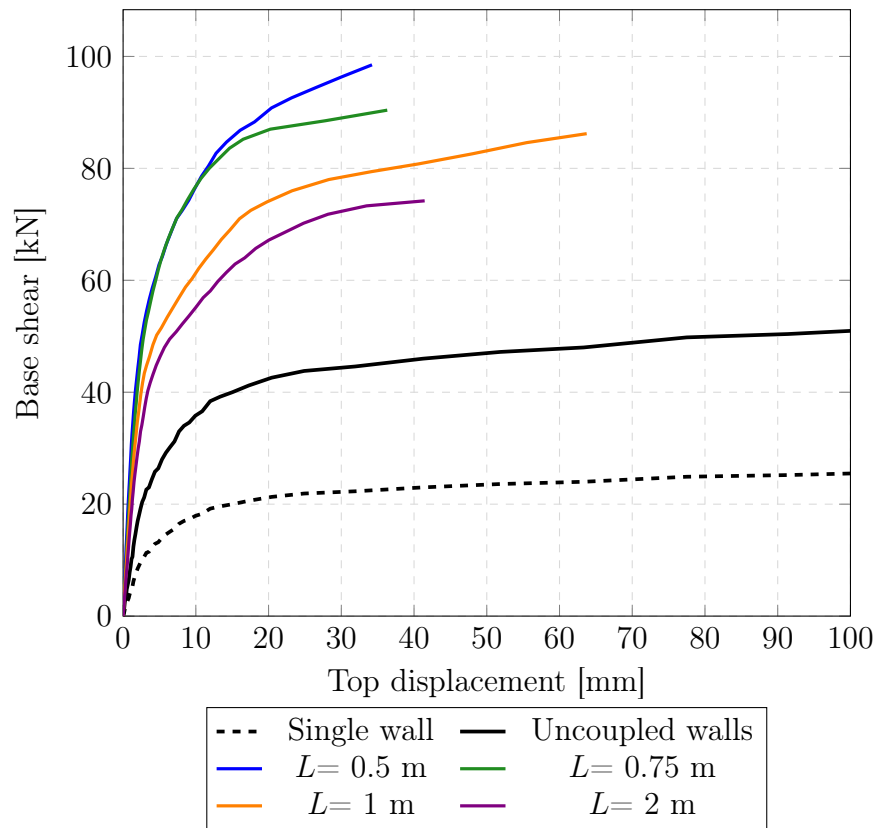


Figure 6.1: Force-displacement curves of slab coupled systems using the real 2 m width from Eq 2.14

the coupled system is significantly higher than the presumed resistance of an uncoupled system constituted of two simple walls. For the shorter 0.5 m slab, the peak resistance of 98.5 kN is almost the double (+87%) of the double wall resistance of 52.8 kN. As expected, increasing the slab length  $L$  is decreasing its stiffness and thus the effectiveness of the coupling connection.

The stiffness is equally increased as the different curves are more steep. The 0.5 m and 0.75 m long slab seem to follow very close paths up to a top displacement of 10 mm and a base shear of 80 kN where they seem to split. The peak resistance is also reached earlier in the coupled configuration.

From the force-displacement curves, the influence of the coupling slabs seems quite clear. It is also possible to analyse the exact contribution of the frame effects to the resistance of the system by looking at the base efforts of the walls and by using the different relations of Figure 2.11.

The additional axial load on the two walls by the slab is of course dependent of the slab stiffness, that is why the shorter 0.5 m slab presents the highest axial load "transfer" at 151 kN. Tension applied by the slab on the wall 1 are actually so high that they overcome the initial axial load, and they actually make the wall work exclusively in tension to around 111 kN. With the increasing length, those transferred forces reduce to 43 kN for

|                  | <b>L=0.5m</b> |       | <b>L=0.75m</b> |       | <b>L=1m</b> |       | <b>L=2m</b> |       |
|------------------|---------------|-------|----------------|-------|-------------|-------|-------------|-------|
|                  | Wall 1        | Wall2 | Wall1          | Wall2 | Wall 1      | Wall2 | Wall1       | Wall2 |
| $N_g$ [kN]       | -40           |       |                |       |             |       |             |       |
| $N_{tot}$ [kN]   | 111           | -190  | 63             | -142  | 35          | 115   | 3           | -83.4 |
| $N_{frame}$ [kN] | 151           | -151  | 103            | -102  | 75          | -75   | 43          | -43   |
| $M_b$ [kNm]      | 25            | 121   | 44             | 110   | 55          | 103   | 63          | 91    |
|                  | -70%          | +42%  | -48%           | +30%  | -35%        | +21%  | -26%        | 7%    |
| $V_b$ [kN]       | 22            | 77    | 24             | 66    | 29          | 56    | 28          | 46    |
|                  | -17%          | +190% | -10%           | +152% | +12%        | +114% | +8%         | +75%  |
| $M_W$ [kNm]      | 146           |       | 155            |       | 158         |       | 153         |       |
|                  | <b>28%</b>    |       | <b>33%</b>     |       | <b>37%</b>  |       | <b>39%</b>  |       |
| $M_F$ [kNm]      | 377           |       | 308            |       | 263         |       | 238         |       |
|                  | <b>72%</b>    |       | <b>67%</b>     |       | <b>63%</b>  |       | <b>61%</b>  |       |
| $M_{tot}$ [kNm]  | 523           |       | 463            |       | 421         |       | 392         |       |
|                  | +208%         |       | +172%          |       | +147%       |       | +130%       |       |

Table 6.3: Efforts at the base for models using the 2 m real width. Italics percentage reflects variation from uncoupled case and bold italics are participation to total base bending moment

the 2 m long slab for which the wall 1 is shows tension reaction of hardly 3kN.

The result of this increase and decrease of axial load is also seen in the base bending moment and shear of the walls. Base bending moment reduction in the tension wall reaches -70% for the shorter slab and comes with a 42% increase for the compression wall. Those numbers decrease as the slab length is augmented, reaching -35% and +21% for the walls of the 1 m long slab model.

Base shear is actually where the largest difference with the uncoupled wall is noticed. The shear demand on the compression wall is almost tripled in the shorter slab configuration, from 26.4 kN to 77 kN. Significant increase is also noticed for the other configurations as for example, the 0.75 m long slab configuration increases by 152% the base shear of wall 2. For the even longer slabs, base shear in the compression wall is of course augmented but it is noticed that whereas it was decreased in the tension wall, it is now increased as well by around 10%.

The participation of the frame effect to the total bending moment resistance is significant and reaches the 72% for the shorter slab configuration. This places it way above

the threshold value defining the coupled systems. All the other configurations exhibit significant participation of the coupling mechanism to the bending resistance, and they could all be categorized as coupled systems.

Those results seem to attest that the slab coupling of shear wall is possible and that it can achieve significant levels of over-strength. However, the hypothesis that the whole slab is contributing to the stiffness and therefore the coupling may seem exaggerated. For slabs with a small length relative to their width, it would be unrealistic to consider that the whole width is actually being solicited. For ratios  $Y/L > 3$ , the increase of width does not appear to have an influence on the slab stiffness [20].

It is therefore imperative to assess the effective width of the slab, which allows for more accurate prediction of the slab coupling mechanism of shear walls.

### 6.3 Reduced effective slab width

Actually, the effective slab width does not depend only on the distance between parallel walls. Other geometric constraints limit this effective width of the slab and therefore its stiffness and ability to act as a coupling element. Equation 2.14 depicts the empirical relationship between the slab length, width and its effective width of [20].

For the same slab length as in the previous section, the effective width of the slab is then adapted to evaluate its ability to still ensure a certain degree of coupling of the shear walls.

| Slab length<br>( $L$ ) [mm] | Slab width<br>( $Y$ ) [mm] | Effective width<br>( $Y_e$ ) [mm] |
|-----------------------------|----------------------------|-----------------------------------|
| 500                         | 2000                       | 450                               |
| 750                         | 2000                       | 650                               |
| 1000                        | 2000                       | 800                               |
| 2000                        | 2000                       | 1200                              |

Table 6.4: Different effective widths

Increasing the slab length and keeping a constant width would only decrease the stiffness of the slab. When considering the effective, this is not as straightforward, the gain of distance between the walls allows for mobilization of more slab. The effective width  $Y_e$  being a function of  $L^2$  and  $L$ , increasing the latter will significantly impact the first. Stiffness being a function of  $L^{-3}$  and  $Y_e$  will therefore be only function of  $L^{-1}$ .

It appears that the slabs are indeed acting as coupling elements in this configuration as well. The differences in force-displacement curves are less obvious than when using the full width of the slabs as effective but there are still some effect of the coupling mechanism.

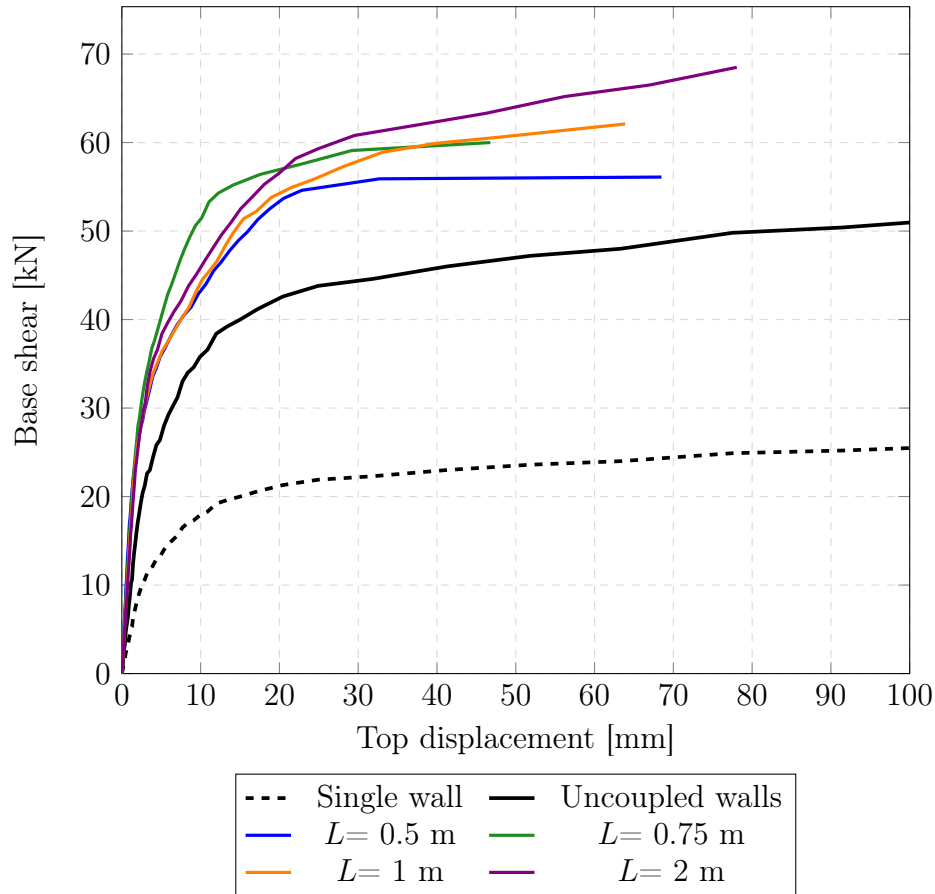


Figure 6.2: Force-displacement curves of slab coupled systems using the effective width from Eq 2.14

All coupled models present an improved lateral load resistance when compared to the uncoupled walls. With the increasing length of the slab and therefore an expected lower stiffness and lower coupling effect, comes an increasing lateral load resistance which may seem surprising. Gain of resistance ranges from +6% for the 0.5 m long slab to up to +30% for the longer 2 m one as can be seen in Table 6.5. The longer and wider slab appears to actually present the highest lateral load resistance among all the tested parameter values. As explained, the increase of length allows for an increase of effective width considered. The peak resistances of the different configurations are thus closer as they present closer stiffnesses.

|                      | $L=0.5$ m | $L=0.75$ m | $L=1$ m | $L=2$ m |
|----------------------|-----------|------------|---------|---------|
| Peak resistance [kN] | 56.1      | 60         | 62.1    | 68.5    |
| Increase             | 6%        | 14%        | 18%     | 30%     |

Table 6.5: Peak resistances of different models

Increase in stiffness can also be observed, improving even more the energy dissipation capacity for a given top displacement. The peak resistance is again reached before the simple uncoupled case.

By looking at the base efforts of both walls, it is possible to evaluate the global impact of the slab coupling using the equilibrium equations of Figure 2.11.

|                  | <b>L=0.5m</b>     |                   | <b>L=0.75m</b>    |                   | <b>L=1m</b>       |                   | <b>L=2m</b>       |                   |
|------------------|-------------------|-------------------|-------------------|-------------------|-------------------|-------------------|-------------------|-------------------|
|                  | Wall 1            | Wall 2            | Wall 1            | Wall 2            | Wall 1            | Wall 2            | Wall 1            | Wall 2            |
| $N_g$ [kN]       | -40               |                   |                   |                   |                   |                   |                   |                   |
| $N_{tot}$ [kN]   | -22               | -58               | -11               | -69               | -12               | 68                | -11               | -69               |
| $N_{frame}$ [kN] | 18                | -18               | 29                | -29               | 28                | -28               | 29                | -29               |
| $M_b$ [kNm]      | 77<br><i>-10%</i> | 92<br><i>+8%</i>  | 70<br><i>-18%</i> | 90<br><i>+5%</i>  | 70<br><i>-18%</i> | 89<br><i>+4%</i>  | 71<br><i>-17%</i> | 90<br><i>+5%</i>  |
| $V_b$ [kN]       | 16<br><i>-39%</i> | 39<br><i>+48%</i> | 22<br><i>-16%</i> | 38<br><i>+44%</i> | 23<br><i>-13%</i> | 39<br><i>+50%</i> | 26<br><i>-3%</i>  | 43<br><i>+60%</i> |
| $M_w$ [kNm]      | 168               |                   | 159               |                   | 158               |                   | 160               |                   |
|                  | <b><i>79%</i></b> |                   | <b><i>65%</i></b> |                   | <b><i>62%</i></b> |                   | <b><i>50%</i></b> |                   |
| $M_F$ [kNm]      | 45                |                   | 87                |                   | 99                |                   | 158               |                   |
|                  | <b><i>21%</i></b> |                   | <b><i>35%</i></b> |                   | <b><i>38%</i></b> |                   | <b><i>50%</i></b> |                   |
| $M_{tot}$ [kNm]  | 213               |                   | 246               |                   | 257               |                   | 319               |                   |
|                  | <i>+25%</i>       |                   | <i>+45%</i>       |                   | <i>+51%</i>       |                   | <i>+87%</i>       |                   |

Table 6.6: Efforts at the base for models using the effective width. Italics percentage reflects variation from uncoupled case and bold italics are participation to total base bending moment

For a length of 0.5 m, the coupling "transfers" 18 kN of axial load from the tension wall to the compression wall. Bending moment and shear are reduced for wall 1 by -10% and -39% while for wall 2 they increase by +8% and +48%. As a result, the frame action contributes to 21% of the total bending resistance. Being obviously smaller than 25% It does not satisfy the Eurocode criteria to be qualified as a coupled system.

Results for the three other slab length quite surprisingly give almost the exact same values. Axial load transfer through the shear deformation of the slab is systematically around 29 kN. The tension wall sees its bending moment reduced by around -18% while its base shear presents more variable reductions from -3% for the 2 m long slab to -16% for the 0.75 m one. Those small variations are to put in perspective with the relatively low values of those efforts. Same observations can be made for the compression wall, as its bending moment demand increases by roughly +5% and its base shear demand is increased by +44% to +60% depending on the slab length.

Base bending moment contribution of the frame mechanism to the total resistance ranges from 21% for the shorter 0.5 m slab to as high as 50% for the longer 2 m one. Given that this longer configuration possesses a longer lever arm for the same frame induced axial load, the bending moment caused by those frame effects is higher. It also means that for lengths superior to 0.75 m, the slab is effectively acting as a coupling element and that the system is to be considered as coupled regarding the Eurocode definition.

Those values appear to be more realistic than the ones obtained by considering the whole width of the slab as effective. Coupling of shear walls through floor slabs has been demonstrated to be possible for the presented configurations.

## 6.4 Axial load ratio

The axial loading of the walls by the coupling slabs through their shear deformations is at the very heart of the coupling mechanism. The impact of more classical axial loading on the coupled system thus seems like an important factor of the coupling effect.

The walls previously studied initially presented an axial load ratio of 2% to mimic the walls tested in [17]. Axial load ratio was then increased to 10% to see what effect it could have on the coupling of the system.

The force displacement curves of 6.3 are all quite similar for all the different slab length, the axial load increase does not seem to have split them apart. This means that the axial load ratio does not influence the sensitivity of the coupling mechanism to different slab stiffness.

The trend observed for the ALR of 2% about the peak resistance increase is still true for this higher ALR. Gains in the range of 3% to 19% were noticed for the different geometries with, again, the longer slab being the most effective.

|                      | $L=0.5$ m | $L=0.75$ m | $L=1$ m | $L=2$ m |
|----------------------|-----------|------------|---------|---------|
| Peak resistance [kN] | 80.1      | 84.3       | 87.5    | 92.2    |
| Increase             | 3%        | 9%         | 13%     | 19%     |

Table 6.7: Peak resistances of different models for ALR of 10%

Again, the lateral stiffness of the system is increased and the peak resistance is reached for a lower top displacement than in the uncoupled case.

Base efforts of the walls are analyzed in order to quantify the coupling effect and their exact contribution to the resisting forces.

Given that the geometry is the same as for the previous case, the stiffness of the slab being the same, the peak axial load transferred by the slabs is situated around 29 kN. All

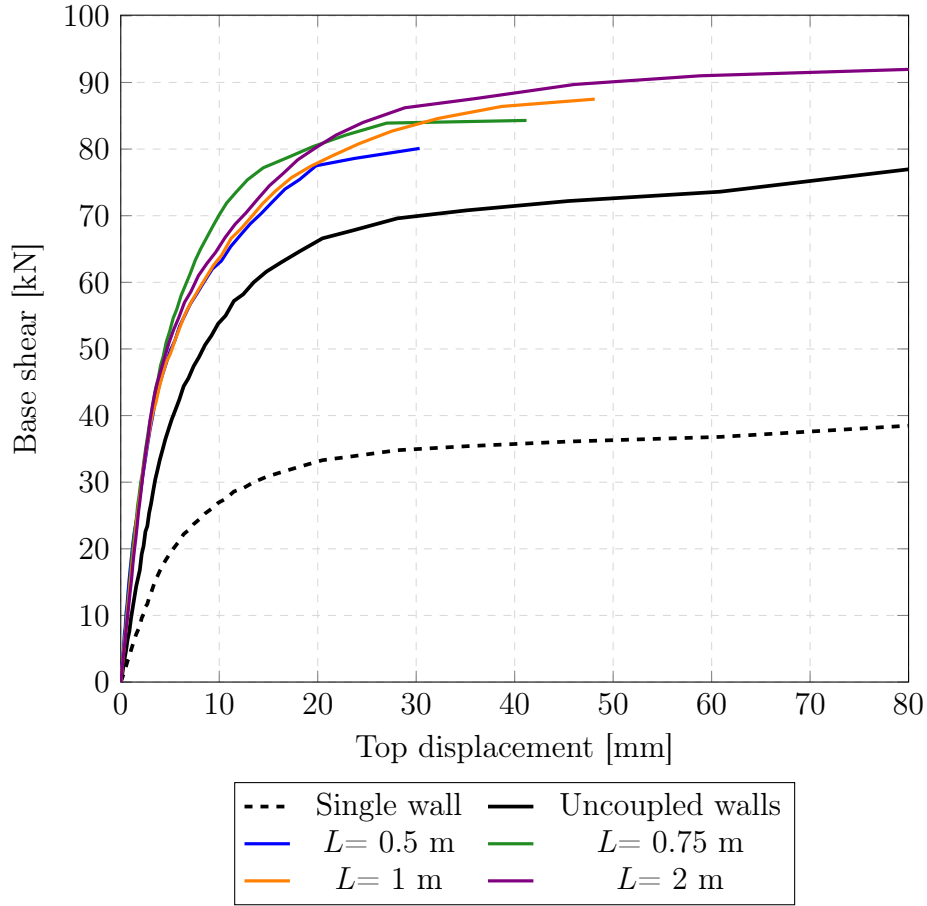


Figure 6.3: Force-displacement curves of slab coupled systems using the effective width from Eq 2.14 for an axial load ratio of 10%

the configurations present the same axial loads of about -166 kN for the tension wall and about -224 kN for the compression wall.

Base bending moments variations are also quite consistent at around -10% and +4% for wall 1 and wall 2 respectively and in the 1 m long slab configuration. Only the longer slab presents bending moment noticeably different from the other models at +1% for the tension and +11% for the compression wall. Base shear demand increase for wall 2 are more important the longer is the slab, with the maximum gain being =53% for the 2 m slab configuration.

As the lever arm between the wall increases and the frame axial load  $N_{frame}$  remains constant, the bending moment due to this frame effect increases. The 0.5 m long slab provided a frame bending moment of 73 kNm, at 23% of the total bending moment, this configuration falls just short of the threshold 25% value which defines a coupled system. This threshold value is matched by the 0.75 m long slab system. The bending moment contribution of the frame effect for the 2 m long slab model reaches 37%, a more than sufficient ratio to allow this system to be considered as coupled.

|                  | <b>L=0.5m</b>      |                   | <b>L=0.75m</b>     |                   | <b>L=1m</b>        |                   | <b>L=2m</b>        |                    |
|------------------|--------------------|-------------------|--------------------|-------------------|--------------------|-------------------|--------------------|--------------------|
|                  | Wall 1             | Wall2             | Wall1              | Wall2             | Wall 1             | Wall2             | Wall1              | Wall2              |
| $N_g$ [kN]       | -195               |                   |                    |                   |                    |                   |                    |                    |
| $N_{tot}$ [kN]   | -166               | -224              | -166               | -222              | -165               | -225              | -168               | -224               |
| $N_{frame}$ [kN] | 29                 | -29               | 29                 | -27               | 30                 | -30               | 27                 | -29                |
| $M_b$ [kNm]      | 109<br><i>-13%</i> | 127<br><i>+1%</i> | 115<br><i>-9%</i>  | 132<br><i>+5%</i> | 114<br><i>-10%</i> | 131<br><i>+4%</i> | 127<br><i>+1%</i>  | 140<br><i>+11%</i> |
| $V_b$ [kN]       | 33<br><i>-15%</i>  | 47<br><i>+22%</i> | 34<br><i>-11%</i>  | 50<br><i>+29%</i> | 36<br><i>-6%</i>   | 51<br><i>+33%</i> | 32<br><i>-17%</i>  | 59<br><i>+53%</i>  |
| $M_w$ [kNm]      | 237                |                   | 247                |                   | 245                |                   | 267                |                    |
|                  | <b><i>77%</i></b>  |                   | <b><i>75%</i></b>  |                   | <b><i>70%</i></b>  |                   | <b><i>63%</i></b>  |                    |
| $M_F$ [kNm]      | 73                 |                   | 84                 |                   | 105                |                   | 155                |                    |
|                  | <b><i>23%</i></b>  |                   | <b><i>25%</i></b>  |                   | <b><i>30%</i></b>  |                   | <b><i>37%</i></b>  |                    |
| $M_{tot}$ [kNm]  | 309                |                   | 331                |                   | 349                |                   | 422                |                    |
|                  | <b><i>+23%</i></b> |                   | <b><i>+25%</i></b> |                   | <b><i>+30%</i></b> |                   | <b><i>+37%</i></b> |                    |

Table 6.8: Efforts at the base for models using the effective width and with an ALR of 10%. Italics percentage reflects variation from uncoupled case and bold italics are participation to total base bending moment

## 6.5 Takeaways

The coupling mechanism of shear wall through reinforced concrete floor slabs has been studied using different finite element models. Starting from a laboratory tested configurations of 4 half-scale walls linked by 3 floor slabs, the influence of these slabs was observed to modify the predicted behaviour of the walls, deviating from the simple cantilever mechanics.

Different slab dimensions were tested using FE software and increase in peak lateral load resistance were predicted to reach up to 30% of resistance in the uncoupled system.

Influence of the length of the slab highlighted the fact that a longer slab could produce significant additional bending moment resistance to the coupled system as the lever arm between the two vertical support reaction is high. This caused the participation of the frame effect induced bending moment resistance to represent as much as 50% of the total resistance, effectively classifying the system as coupled according to the EN1998-1 definition of coupled systems. Increase of sometimes as much as 60% of the bending moment demand was also observed in the compression wall

The case has therefore been made that slab coupling of reinforced concrete shear wall should be considered when evaluating the response of a building to horizontal solicitation.

Improvement on the models used for this analysis could mainly reside in the slab modelling. Using a 3D FE software could also allow to more precisely compute the effective width of the slab and not rely on an empirical formula. More values of length and width could be studied as well as the thickness of the slab which is an important factor of its stiffness. Finally, different shapes of wall section could also be investigated, as flanged walls for example would help engage more of the slab and increase the effect of coupling.



## Chapter 7

# Conclusion and outlook

## 7.1 Conclusion

Two different wall-diaphragm interactions were studied in this thesis. The coupling of shear walls with floor slabs and the "stirrup effect". The numerical analysis of nearly 50 finite elements models was conducted and detailed in Chapter 5 and 6.

### 7.1.1 Stirrup effect

This interaction between wall and floor diaphragm was investigated as it was expected to modify the shear behaviour of the wall. Having a discrete restraint applied on its section which would prevent it from expanding and prevent cracks from opening seems like a good way to prevent or delay any shear failure.

A parametric study of Chapter 5 tested different values of parameters such as the axial load ratio, transverse and longitudinal reinforcements or aspect ratio. It showed that the impact of the floor diaphragm was quite limited for the different values tested, as the difference in resistance was non-existent and the difference in displacement capacity was often negligible and showed no trend correlated to the different parameters studied.

A more targeted analysis was then conducted to more closely study the impact of the reinforcement ratios on the stirrup effect. For very low  $\rho_t/\rho_l$  ratios, an increase in lateral load resistance was actually observed up to 12%.

### 7.1.2 Slab coupling

The beam coupling of walls being already defined, the apparent weaker stiffness of concrete slabs is often regarded as a good reason to not consider the floor diaphragm as a coupling element. Different studies and laboratory tests have however shown that slab coupling of reinforced concrete shear wall is possible.

Again, a finite element analysis was conducted to determine if slab could effectively induce coupling forces into a wall system. The stiffness of the slab being the main factor in the efficient coupling of the walls, different factors influencing this stiffness were tested. Namely the slab length and its width. While the length decreases the bending stiffness of the slab, the width increases it. An effective width empirical formula was also used in the simulations.

The resistance of slab coupled walls increased by as much as 30% compared to the uncoupled case where they were supposed to work independently.

According to the Eurocode 8 definition of a coupled system, most of the studied configurations would need to be classified as coupled. Thus, highlighting the importance to consider the possible coupling of shear walls through reinforced concrete slabs.

## 7.2 Outlook

Further research on those subject would be welcomed. Improving the finite elements modelling by finding a way to impose two or more related displacement would allow discarding the additional structure of Chapter 5 and to use a displacement driven analysis to capture the post-peak behaviour of the coupled systems. Solution to the localization issues, such as regularization, being used could allow for considering the concrete strain hardening model by Bentz and better capture the tensile behaviour of concrete. The effective width of the slab could also be researched, as the hypothesis of infinite stiffness used here is quite conservative and may not reflect the real behaviour of a floor diaphragm. Models or test could as well be developed to evaluate this effective width.

While the results regarding the stirrup effect were not highly compelling, the relative variability in the results may make them seem somewhat unreliable. Real life laboratory testing would be desirable to corroborate or invalid the findings of this thesis.

The slab coupling has, however, shown a real potential impact on the behaviour of walls. The effective width of the slab could be better captured using a 3D FE software. Displacement based approach could also allow to assess the post-peak behaviour and the possible ductility change as well as a clearer view on the failure modes. Again different geometrical values can be tested such as walls spacing and section shapes.

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## Appendix A

### Chapter 1 additional information

## Deformation and resistance characteristics of concrete

| Classes de résistance du béton |      |     |     |     |     |      |     |     |      |      |     |      |     |     | Expression analytique<br>Commentaires                                                                         |
|--------------------------------|------|-----|-----|-----|-----|------|-----|-----|------|------|-----|------|-----|-----|---------------------------------------------------------------------------------------------------------------|
| $f_{ck}$<br>(MPa)              | 12   | 16  | 20  | 25  | 30  | 35   | 40  | 45  | 50   | 55   | 60  | 70   | 80  | 90  |                                                                                                               |
| $f_{ck,cube}$<br>(MPa)         | 15   | 20  | 25  | 30  | 37  | 45   | 50  | 55  | 60   | 67   | 75  | 85   | 95  | 105 |                                                                                                               |
| $f_{cm}$<br>(MPa)              | 20   | 24  | 28  | 33  | 38  | 43   | 48  | 53  | 58   | 63   | 68  | 78   | 88  | 98  | $f_{cm} = f_{ck} + 8$ (MPa)                                                                                   |
| $f_{ctm}$<br>(MPa)             | 1,6  | 1,9 | 2,2 | 2,6 | 2,9 | 3,2  | 3,5 | 3,8 | 4,1  | 4,2  | 4,4 | 4,6  | 4,8 | 5,0 | $f_{ctm} = 0,30 \times f_{ck}^{(2/3)} \leq C50/60$<br>$f_{ctm} = 2,12 \cdot \ln(1 + (f_{cm}/10))$<br>> C50/60 |
| $f_{ctk,0,05}$<br>(MPa)        | 1,1  | 1,3 | 1,5 | 1,8 | 2,0 | 2,2  | 2,5 | 2,7 | 2,9  | 3,0  | 3,1 | 3,2  | 3,4 | 3,5 | $f_{ctk,0,05} = 0,7 \times f_{ctm}$<br>fractile 5 %                                                           |
| $f_{ctk,0,95}$<br>(MPa)        | 2,0  | 2,5 | 2,9 | 3,3 | 3,8 | 4,2  | 4,6 | 4,9 | 5,3  | 5,5  | 5,7 | 6,0  | 6,3 | 6,6 | $f_{ctk,0,95} = 1,3 \times f_{ctm}$<br>fractile 95 %                                                          |
| $E_{cm}$<br>(GPa)              | 27   | 29  | 30  | 31  | 33  | 34   | 35  | 36  | 37   | 38   | 39  | 41   | 42  | 44  | $E_{cm} = 22[(f_{cm})/10]^{0,3}$<br>( $f_{cm}$ en MPa)                                                        |
| $\varepsilon_{c1}$<br>(‰)      | 1,8  | 1,9 | 2,0 | 2,1 | 2,2 | 2,25 | 2,3 | 2,4 | 2,45 | 2,5  | 2,6 | 2,7  | 2,8 | 2,8 | Voir figure 3.2<br>$\varepsilon_{c1}(\text{‰}) = 0,7 f_{cm}^{0,31} \leq 2,8$                                  |
| $\varepsilon_{cu1}$<br>(‰)     | 3,5  |     |     |     |     |      |     |     |      | 3,2  | 3,0 | 2,8  | 2,8 | 2,8 | Voir figure 3.2<br>pour $f_{ck} \geq 50$ MPa<br>$\varepsilon_{cu1}(\text{‰}) = 2,8 + 27[(98 - f_{cm})/100]$   |
| $\varepsilon_{c2}$<br>(‰)      | 2,0  |     |     |     |     |      |     |     |      | 2,2  | 2,3 | 2,4  | 2,5 | 2,6 | Voir Figure 3.3<br>pour $f_{ck} \geq 50$ MPa<br>$\varepsilon_{c2}(\text{‰}) = 2,0 + 0,085(f_{ck} - 50)^{0,5}$ |
| $\varepsilon_{cu2}$<br>(‰)     | 3,5  |     |     |     |     |      |     |     |      | 3,1  | 2,9 | 2,7  | 2,6 | 2,6 | Voir Figure 3.3<br>pour $f_{ck} \geq 50$ MPa<br>$\varepsilon_{cu2}(\text{‰}) = 2,6 + 35[(90 - f_{ck})/100]$   |
| $n$                            | 2,0  |     |     |     |     |      |     |     |      | 1,75 | 1,6 | 1,45 | 1,4 | 1,4 | pour $f_{ck} \geq 50$ MPa<br>$n = 1,4 + 23,4[(90 - f_{ck})/100]^4$                                            |
| $\varepsilon_{c3}$<br>(‰)      | 1,75 |     |     |     |     |      |     |     |      | 1,8  | 1,9 | 2,0  | 2,2 | 2,3 | Voir Figure 3.4<br>pour $f_{ck} \geq 50$ MPa<br>$\varepsilon_{c3}(\text{‰}) = 1,75 + 0,55[(f_{ck} - 50)/4]$   |
| $\varepsilon_{cu3}$<br>(‰)     | 3,5  |     |     |     |     |      |     |     |      | 3,1  | 2,9 | 2,7  | 2,6 | 2,6 | Voir Figure 3.4<br>pour $f_{ck} \geq 50$ MPa<br>$\varepsilon_{cu3}(\text{‰}) = 2,6 + 35[(90 - f_{ck})/100]$   |

Figure A.1: VecTor2 nonlinear finite element analysis algorithm - from [34]

## Finite element analysis algorithm

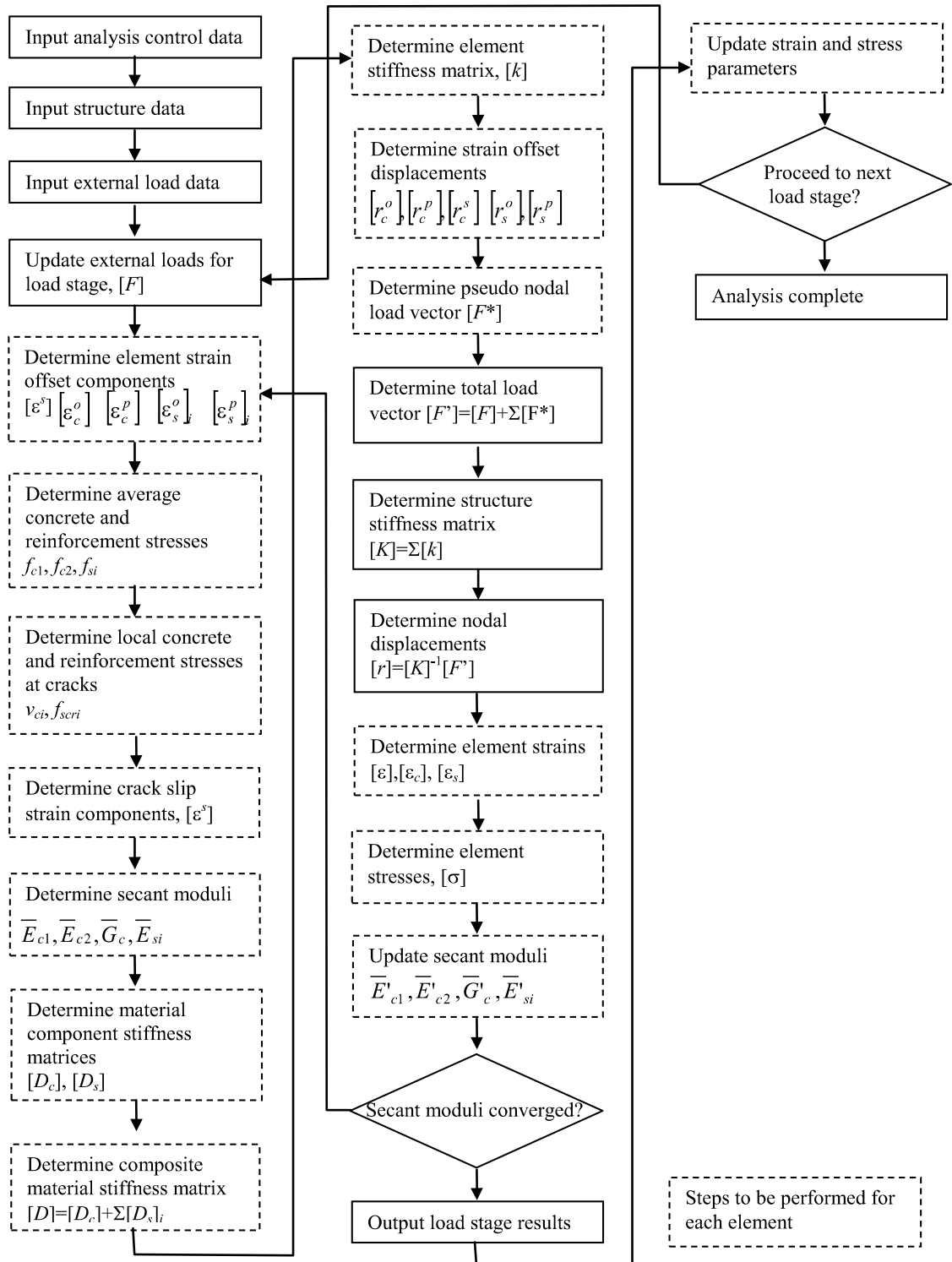


Figure A.2: VecTor2 nonlinear finite element analysis algorithm - from [30]



## Appendix B

### Stirrup effect simulations

#### B.1 Targeted analysis

B.1.1  $\rho_l = 2.00\%$ ;  $ALR = 25\%$ ;  $AR = 1.85$

$$\rho_t = 0.10\% \quad ; \quad \rho_t/\rho_l = 5\%$$

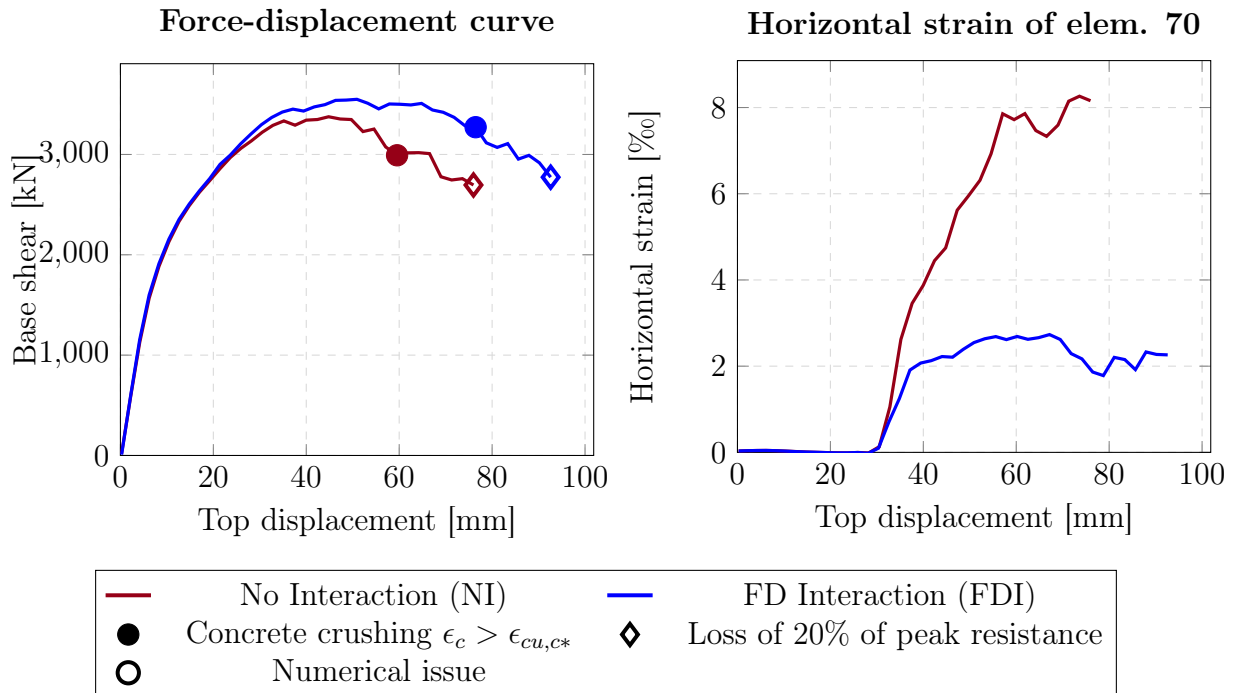
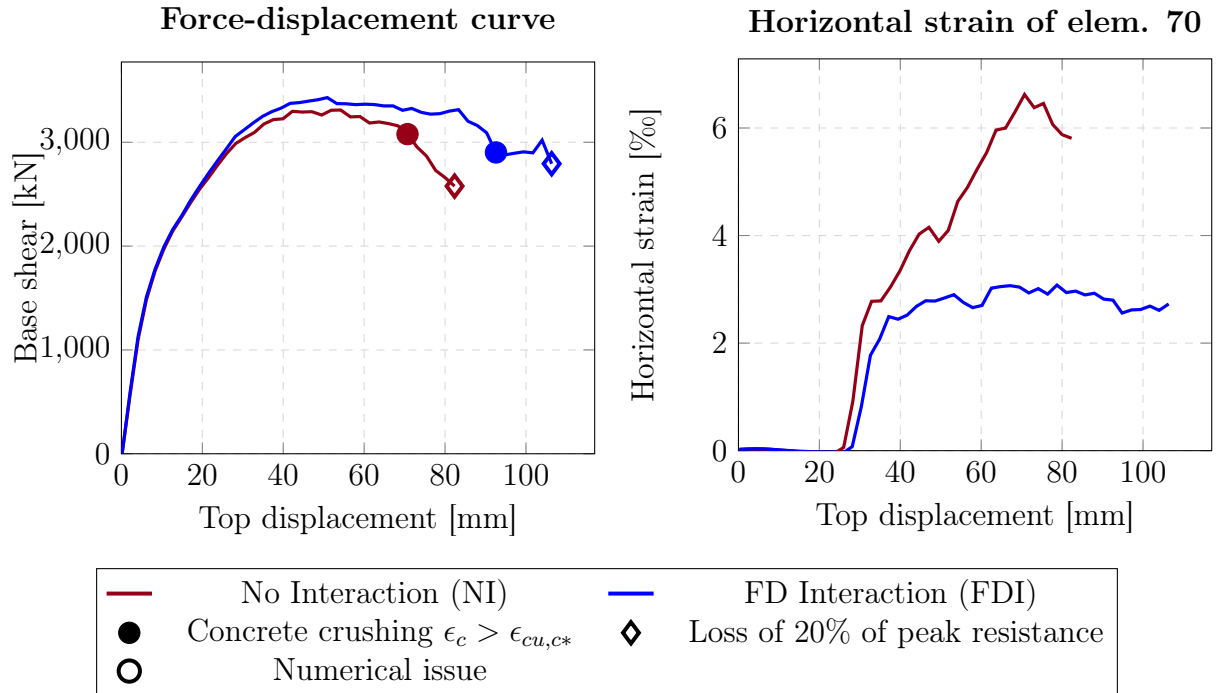


Figure B.1: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 2\%$  ;  $\rho_t = 0.10\%$  ;  $ALR = 25\%$

|                            | NI model | FDI model  |
|----------------------------|----------|------------|
| Peak resistance [kN]       | 3375     | 3549 (+5%) |
| Displacement capacity [mm] |          |            |
| - Concrete crushing        | 60       | 76 (+28%)  |
| - Resistance degradation   | 76       | 93 (+22%)  |

Table B.1: Changes in resistance and displacement capacity for  $\rho_t = 0.1$ 

$$\rho_t = 0.15\% \quad ; \quad \rho_t/\rho_l = 8\%$$

Figure B.2: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 2\%$  ;  $\rho_t = 0.15\%$  ; ALR = 25%

|                            | NI model | FDI model  |
|----------------------------|----------|------------|
| Peak resistance [kN]       | 3311     | 3430 (+4%) |
| Displacement capacity [mm] |          |            |
| - Concrete crushing        | 71       | 93 (+31%)  |
| - Resistance degradation   | 82       | 106 (+22%) |

Table B.2: Changes in resistance and displacement capacity for  $\rho_t = 0.15$

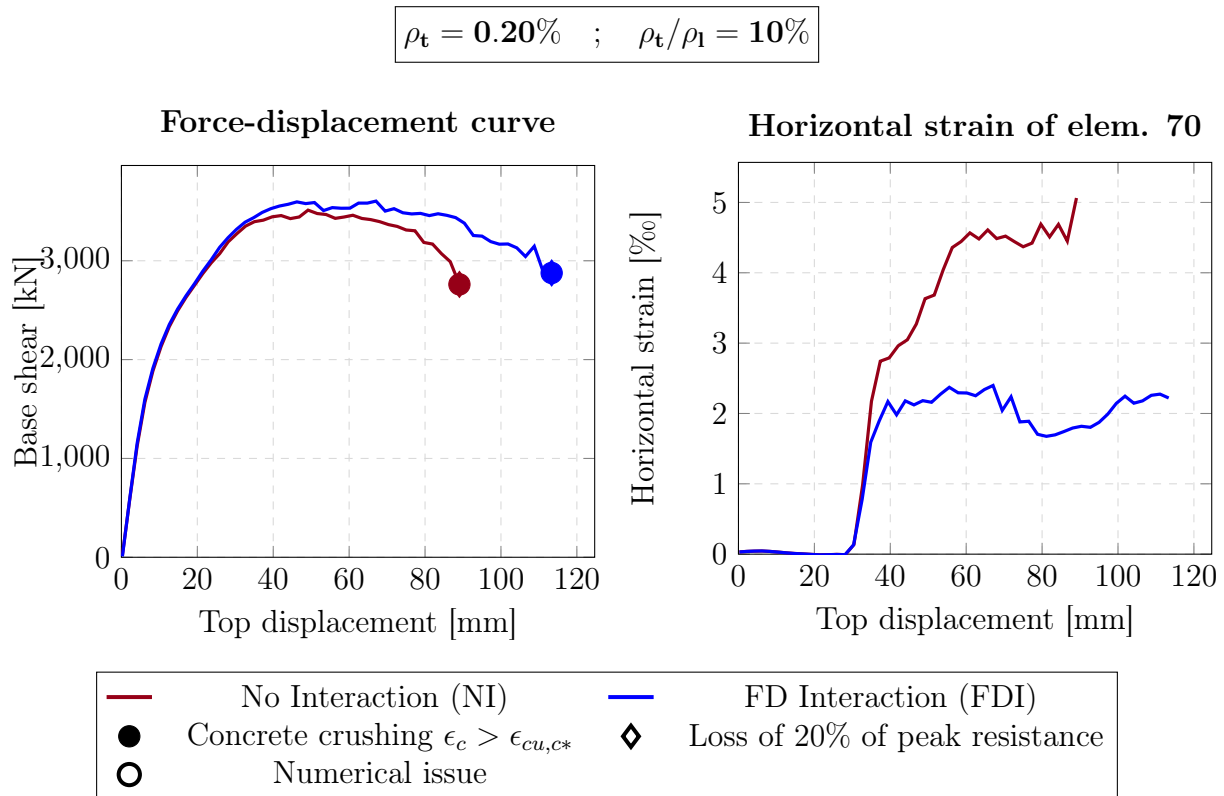


Figure B.3: Force-displacement curve and element no. 70 horizontal strain for models of configuration :  $\rho_l = 2\%$  ;  $\rho_t = 0.20\%$  ; ALR = 25%

|                            | NI model | FDI model  |
|----------------------------|----------|------------|
| Peak resistance [kN]       | 3512     | 3603 (+3%) |
| Displacement capacity [mm] |          |            |
| - Concrete crushing        | 89       | 113 (+27%) |
| - Resistance degradation   | 89       | 113 (+27%) |

Table B.3: Changes in resistance and displacement capacity for  $\rho_t = 0.20$

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