Seismic risk assessment and mitigation for Belgium in the frame of EUROCODE 8

Programme d'appui scientifique à la normalisation et aux réglementations techniques

Rapport final

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André PLUMIER & Catherine DONEUX

Denis JONGMANS, Marc WATHELET, Hervé TEERLYNCK & Fredéric NGUYEN

Thierry CAMELBEECK & Gilles VAN ROMPAEY

UNIVERSITY OF LIEGE
M&S DEPARTMENT (MECHANICS OF MATERIALS AND STRUCTURES)
MECHANICAL AND CIVIL ENGINEERING INSTITUTE, BUILDING B52
Allée des chevreuils 1
B-4000 LIEGE

UNIVERSITY OF LIEGE
LABORATORIES OF ENGINEERING GEOLOGY, HYDROGEOLOGY, AND GEOPHYSICAL PROSPECTING
MECHANICAL AND CIVIL ENGINEERING INSTITUTE, BUILDING B52
Allée des chevreuils 1
B-4000 LIEGE

ROYAL OBSERVATORY OF BELGIUM
Avenue Circulaire 3
B-1180 BRUXELLES

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SUMMARY

A. Context

To be allowed to use the Eurocodes, each country has to produce a National Application Document (NAD) in which particular elements are fixed under the authority of national bodies. In 2000, the Belgian regulation concerning the seismic design of structures was not extensively developed: imprecision on the action to apply to the structures, lack of knowledge on the vulnerability of typical Belgian housing with respect to low seismicity context, lack of knowledge on an adapted art of building.

B. Objectives

The present project, which gathers geologists, seismologists and engineers, aims at defining a common point of view in the Belgian earthquake engineering community.

The main global objective of the project was to define in a short-term period, applicable seismic regulations which are suitable for the seismic hazard and vulnerability in Belgium within the frame of EUROCODE-8.

A second objective was to define a common Belgian position on the various questions arising in EUROCODE-8, in particular a correct seismic action definition (elastic response spectrum), site-specific spectra shapes and practical building regulations adapted to low seismicity regions like Belgium.

To reach these goals, it was necessary:

- To determine the source characteristics of the reference earthquakes to take into account in Belgium for a return period of 475 years.
- To study the site effects in different regions of Belgium and to specify their characteristics for typical geological conditions in Belgium. This is done by in-site experiments and computations from geological, geotechnical and geophysical data.
- To develop and evaluate the capabilities of fast and low cost field measurements to evaluate site effects.
- To define the design response spectra for rock and soil corresponding to the different seismic zones distinguished in the regional seismic hazard map.
- To compare the results with the present proposals for spectra shape modifications and to define a common National point of view for the representatives in the European commissions.
- To analyse by 3D modelling typical Belgian simple building structures under low earthquake action
- To deduce from these analyses the connection forces between constructional elements (walls, floors, roofs) and the action effects in these elements themselves.
- To design and sketch the connection devices and their lay out in the various critical parts of the buildings and to produce design rules for end users concerning these simple structures.
- To disseminate the obtained results at the National level.
- To implement the obtained results in EUROCODE 8 through the natural channel constituted by the participation of one member of the research network in the Project Team of EUROCODE 8.

C. Conclusions

Concerning the definition of the seismic action to apply to engineering structures, this study is the first ever conducted which is based on the real seismic context of Belgium. It completes the seismic hazard map on the bedrock already realized in the framework of EUROCODE-8 by two fundamental aspects: (a) the definition of elastic response spectra for two different reference earthquakes which should allow to realize a choice for the National Document of Application for EUROCODE-8; and (b) the evidence that regional site effects in the north of the country should be taken into account by a convenient choice of the response spectra. It appears also that the region of the Mons basin should be studied more intensively in the future, because the study suggests that site effects have there disastrous effects on the strong ground motions.

Concerning the practical application of building regulations, two types of results are obtained for the vulnerability study of non-engineered structures in low seismic activity zones as Belgium:

- Values of the connection forces between constructional elements (walls, floors, roofs). The values to be considered in the design of the links between the constitutive elements are explained and accompanied by recommendations.
- Drawing of connecting details between constitutive parts and their layout in the various critical parts of the buildings.

The interest of the obtained results is that they provide an estimate of the forces to take into account for the seismic design of usual masonry structures like dwellings. Coming from a situation in which no design value existed, these results are a serious step ahead and will for sure be very useful. The results are presented in a technical guide, which does not only give drawings, but also summarises in a simple way the basic seismic design principles to allow the reader to understand and use the results in a wise manner. The further step is to convert this technical guide and all the collected knowledge about seismic behaviour of structures in low seismicity regions into a bi-lingual technical handbook. This handbook would be edited by the BBRI (Belgian Building Research Institute) in order to achieve the widest dissemination thanks to the existing channels of this Institution, which gather all Belgian constructions companies of all size.

D. Contribution of the project in a context of support to the processes of standardisation and technical regulations

The coordinator of the project is also the chairman of the Belgian Committee in charge of discussing and producing Technical Annexes to the EUROCODE-8. As soon as the relevant sections of EUROCODE-8 will become a Euronorm, which should take place in the summer of 2003, the Committee will meet to achieve the Technical Annex. Contacts with similar committees in neighbour countries have also recently taken place.

On the basis of all the results obtained in the project, a pre-standard document will be written. It will constitute a new release of the National Document of Application for EUROCODE-8. On another hand, a draft will also be proposed for the parts of EUROCODE-8, which are debated. The writing will be focused on a clear and unequivocal definition of the concepts and the regulations to follow in countries with a low seismic activity, in agreement with the follow up committee. This will ensure to obtain a document understandable for all the people involved in earthquake engineering and applicable for all the users.

Besides of the strict activity of preparing code documents, a first seminar open to architects, engineers and construction companies took place in February 2003. It will be reconducted in the end of 2003.

E. Keywords

seismic – earthquake – Belgium – EUROCODE-8 – masonry non-engineered structures – low seismicity – site effects.

RÉSUMÉ

A. Contexte

Pour pouvoir utiliser les Eurocodes, chaque pays doit produire des documents d'application nationaux (DAN) dans lesquels des éléments particuliers sont fixés par les autorités nationales. En 2000, la réglementation belge concernant le dimensionnement sismique des structures n'était pas entièrement définie : imprécision sur les actions à appliquer aux structures, manque d'information sur la vulnérabilité de l'habitat type belge par rapport à une sismicité faible, manque d'information sur un art de construire adapté.

B. Objectifs

Le présent projet, qui réunit des géologues, sismologues et ingénieurs, a pour but de définir un point de vue commun dans la communauté belge en génie parasismique.

L'objectif global principal de ce projet de recherche est de définir à court terme, dans le cadre de l'Eurocode 8, une réglementation sismique applicable et adaptée à la situation belge (aléa sismique et vulnérabilité).

Le second objectif principal du projet de recherche est de définir une position belge face aux questions posées au sein de l'Eurocode 8, à la fois au niveau d'une définition correcte de l'action sismique, des formes de spectres de réponse dépendant des conditions de site et de l'application de normes constructives, en particulier celles concernant les constructions "non-ingéniérées" (maisons, petits bâtiments) dans des zones de relativement faible séismicité.

Les principales étapes nécessaires pour atteindre ces objectifs sont :

- Déterminer les caractéristiques de sources pour les séismes dangereux en Belgique.
- Étudier les effets de site dans différentes zones et spécifier leurs caractéristiques générales sur l'ensemble du pays. Cette analyse est réalisée à partir d'expériences in situ et de calculs basés sur des données géologiques et géophysiques existantes.
- Définir les spectres de réponse de référence sur le rocher et sur sols pour les zones sismiques définies sur la carte d'aléa sismique régional.
- Comparer les résultats obtenus avec les propositions actuelles de modification de forme des spectres de l'EC8 et définir une position nationale pour les représentants dans les comités européens.
- Analyser par modélisation 3D le comportement dynamique de constructions belges simples et typiques.
- Déduire de cette analyse les efforts dans les connexions entre les éléments de constructions (murs, planchers, toitures) et les sollicitations dans ces éléments eux-mêmes.
- Concevoir, dimensionner et dessiner les connexions dans les parties critiques des bâtiments ainsi que produire des règles de dimensionnement pour les utilisateurs finaux, architectes, entrepreneurs, concernant ces structures simples.
- Diffuser les résultats obtenus au niveau national.
- Introduire les résultats obtenus dans l'EUROCODE 8 par l'intermédiaire du membre du réseau faisant partie du "Project Team" de l'EUROCODE 8.

C. Conclusions

En ce qui concerne la définition de l'action sismique à appliquer aux structures, cette étude est la première à être réalisée en se basant sur le contexte sismique belge réel. Elle complète la carte d'aléa sismique sur le rocher déjà réalisée dans le cadre de l'EUROCODE 8 par deux aspect fondamentaux : (a) la définition des spectres de réponse élastique pour deux tremblements de terre de référence différents, qui devrait permettre de faire un choix au niveau du Document d'Application National de l'EUROCODE-8 ; et (b) la preuve que les effets de site régionaux du nord du pays devraient être pris en compte par un choix adéquat des spectres de réponse. Il apparaît aussi que la région du bassin de Mons devrait être plus intensivement étudiée dans le futur, car l'étude suggère que dans cette région, les effets de site ont un effet désastreux sur les mouvements de sol importants.

En ce qui concerne l'application pratique des règles de dimensionnement des bâtiments, deux types de résultats ont été obtenus à partir de l'étude de vulnérabilité des structures non ingéniérées dans des zones de faible intensité sismique comme la Belgique :

- des valeurs des forces de liaison entre les éléments constitutifs de la construction (murs, planchers, toiture). Les valeurs à considérer dans le dimensionnement des liaisons entre éléments constitutifs sont expliquées et accompagnées de recommandations.
- des dessins des détails constructifs de liaison entre éléments constitutifs et leur positionnement dans les diverses parties critiques des bâtiments.

L'intérêt des résultats obtenus est qu'ils fournissent une estimation des efforts à considérer dans le dimensionnement parasismique des structures simples en maçonnerie telles que les maisons d'habitation privées. Venant d'une situation existante sans valeur de dimensionnement, ces résultats représentent un sérieux pas en avant et seront sans aucun doute d'une grande utilité. Les résultats sont présentés dans un guide technique, qui ne donne pas uniquement des dessins de détails constructifs, mais qui résume aussi de manière simplifiée, les principes de base du dimensionnement parasismique pour permettre au lecteur de comprendre et d'utiliser les résultats de manière avisée. L'étape suivante est de convertir ce guide technique et toutes les connaissances collectées sur le comportement sismique des structures dans les régions de faible sismicité dans un manuel technique bilingue. Ce manuel serait édité par le CSTC (Centre Scientifique et Technique de la Construction) afin de réaliser la plus large distribution possible grâce aux canaux existants de cette Institution qui regroupe toutes les entreprises de construction belges, petites et grandes.

D. Apport du projet dans un contexte d'appui aux processus de normalisation et de réglementations techniques

Le coordinateur du projet est aussi le président du Comité belge en charge de la discussion et de la production des Annexes Techniques de l'EUROCODE-8. Dès que les parties utiles de l'EUROCODE-8 seront transformées en Euronorme, ce qui devrait avoir lieu au cours de l'été 2003, le Comité se réunira pour terminer l'Annexe Technique. Des contacts avec des comités similaires des pays voisins ont aussi récemment été pris.

Sur base de tous les résultats obtenus dans le cadre du projet, une pré-norme sera écrite. Elle constituera une nouvelle version du Document d'Application National de l'EUROCODE-8. D'autre part, une première version (draft) sera proposée pour les parties de l'EUROCODE-8 qui sont débattues. La rédaction se focalisera sur une définition claire et univoque des concepts et de la réglementation à appliquer dans les pays à faible séismicité, en accord avec le comité d'accompagnement du réseau. Cela garantira l'obtention d'un document compréhensible par l'ensemble des acteurs impliqués dans l'ingénierie sismique et applicable par tous les utilisateurs.

A côté de l'activité de préparation de documents de code, un premier séminaire ouvert aux architectes, aux ingénieurs et aux entreprises de construction a été organisé en février 2003. Un autre séminaire du même type sera organisé fin 2003.

E. Mots-clés

sismique – tremblement de terre – Belgique – EUROCODE-8 – maçonnerie - structures non ingéniérées – faible séismicité – effets de site.

SAMENVATTING

A. Context

Om een toelating te krijgen de EUROCODE te gebruiken, moet ieder land een Nationaal Toepassingsdocument (NTD) opstellen waarin specifieke elementen zijn vastgelegd door de bevoegde nationale overheden. In 2000 waren de voorschriften voor structurele ontwerpen niet volledig ontwikkeld: er was onduidelijkheid over de op de structuren toe te passen acties, gebrek aan kennis over de kwetsbaarheid van een typisch Belgische bewoning met betrekking tot een lage seismiciteit en een gebrek aan kennis over een aangepaste bouwwijze.

B. Doelstellingen

Het huidige project brengt geologen, seismologen en ingenieurs samen met de bedoeling een gemeenschappelijk standpunt te definiëren voor de antiseismische bouwkunde in België. Het hoofddoel van het project was voorschriften op te stellen die op korte termijn kunnen toegepast worden en aangepast aan de kans op en kwetsbaarheid voor aardbevingen in België binnen het kader van EUROCODE-8.

Een tweede doel betrof het opstellen van gemeenschappelijk Belgisch standpunt over verscheidene vragen in EUROCODE-8, in het bijzonder de definitie van een juiste seismische actie (elastic response spectrum), plaatsgebonden spectra en praktische bouwvoorschriften aangepast aan gebieden met een lage seismiciteit zoals in België.

Om dit te bereiken, was het nodig:

- De karakteristieken te bepalen van de haard van een referentie aardbeving die diende in rekening gebracht te worden voor een terugkeerperiode van 475 jaar.
- De lokale bodemeffecten te bestuderen in verschillende gebieden van het land en de karakteristieken te specificeren voor typische geologische omstandigheden. Dit werd gedaan door in-situ experimenten en berekeningen van geologische, geotechnische en geofysische gegevens.
- De mogelijkheden te ontwikkelen en te evalueren van snelle en goedkope veldmetingen om lokale bodemeffecten op te meten.
- Om de referentie “response spectra” te definiëren voor een rotsbodem en voor bodems die overeenstemmen met de verschillende seismische zones in de regionale kaart met de kans op aardbevingen.
- Om de resultaten te vergelijken met de huidige voorstellen voor wijzigingen in de modellen van spectra en om een gemeenschappelijk standpunt te definiëren ten behoeve van de nationale vertegenwoordigers in de Europese Commissie.
- Om eenvoudige typische drie-dimensionele Belgische bouwstructuren te analyseren bij een lage aardbevingsactie.
- Om uit deze analyses de bindingskrachten af te leiden tussen de constructie-elementen (muren, vloeren, daken), en de effecten van de acties in deze elementen zelf.
- Om de bindingsstukken te ontwerpen, uit te tekenen en hun lay-out in de verschillende kritische gedeelten van de gebouwen en ontwerpvoorschriften op te stellen voor de eindgebruikers van deze eenvoudige structuren.
- Om de verkregen resultaten te verspreiden op nationaal niveau.
- Om de resultaten toe te passen op EUROCODE 8 door de deelname van een lid van het onderzoeksnetwerk in het EOROCODE 8 Project Team.

C. Besluiten

Met betrekking tot de definitie van een seismische actie die op een structuur dient toegepast te worden, is dit de eerste studie die ooit is uitgevoerd op basis van de werkelijke seismische context in België. De kaart met de kans op aardbevingen voor rotsbodem, opgemaakt in het kader van EUROCODE 8, wordt aangevuld met twee belangrijke aspecten: (a) de definitie van elastische response spectra voor twee verschillende referentie aardbevingen wat het mogelijk moet maken een keuze te maken voor het Nationaal Toepassingsdocument voor EUROCODE 8; en (b) het bewijs dat men in het Noorden van het land moet rekening houden met regionale lokale bodemeffecten door een aangepaste keuze van de response spectra. Het blijkt ook dat de streek rond Mons (Bergen) in de toekomst meer intensief zou moeten bestudeerd worden omdat uit de studie blijkt dat de lokale bodemeffecten een rampzalig effect hebben op hevige bodembeweging.

Met betrekking tot de praktische toepassingen van de bouwvoorschriften werden er twee resultaten bekomen voor de kwetsbaarheid van niet-geëngineerde structuren in zones met een lage seismiciteit zoals België:

Waarden voor de bindingskrachten tussen constructie-elementen (muren, vloeren,daken). De waarden die men dient te gebruiken in het ontwerp van bindingen tussen de samenstellende elementen worden verklaard en vergezeld met aanbevelingen.

Tekeningen van details voor de bindingen tussen de samenstellende delen en hun lay-out in de verschillende kritische delen van een gebouw.

Het belang van de verkregen resultaten ligt in de schatting van de krachten waarmee men moet rekening houden voor een seismisch ontwerp van gewone bakstenen structuren zoals woonhuizen. Komende van een situatie waarin geen referentiewaarden bestonden, vormen deze resultaten een ernstige stap voorwaarts en zullen zeer zeker nuttig zijn. De resultaten worden voorgesteld in een technische gids, die niet alleen tekeningen bevat, maar ook op een eenvoudige manier de basisprincipes samenvat van een seismisch ontwerp om het de lezer mogelijk te maken de resultaten op een verstandige manier toe te passen. De volgende stap zal er in bestaan om deze technische gids en alle verzamelde kennis over het seismisch gedrag van structuren in zones met een lage seismiciteit om te zetten in een tweetalig technisch handboek. Dit handboek zal uitgegeven worden door het WTCB (Wetenschappelijk en Technisch Centrum voor het Bouwbedrijf) om op die manier de grootst mogelijke spreiding te bekomen dank zij de banden die dit instituut bindt met alle Belgische bouwondernemingen van welke grootte dan ook.

D. Bijdrage van het project in een context van ondersteuning aan de processen inzake normalisatie en technische regelgeving

De coördinator van het project is ook de voorzitter van de Belgische commissie die belast is met de bespreking en de opstelling van de Technische Bijlages van EUROCODE-8. Van zodra de relevante delen van EUROCODE-8 een Euronorm zullen worden, wat voorzien is in de zomer van 2003, zal de Commissie bijeenkomen om de Technische Bijlage af te werken. Recent zijn ook contacten gelegd met gelijkaardigde commissies in de buurlanden.

Op basis van alle resultaten die in het project zijn bekomen, zal een pre-standaard document opgesteld worden. Dit zal een nieuwe uitgave worden van het Nationaal Toepassingsdocument voor EUROCODE-8. Anderzijds zal ook een ontwerptekst voorgesteld worden voor de nog te bespreken delen van EUROCODE-8. De tekst zal ingesteld worden op een duidelijke en ondubbelzinnige definitie van de concepten en voorschriften die te volgen zijn in landen met een lage seismische activiteit en dit in overeenstemming met de opvolgingscommissie. Dit zal het mogelijk maken om een document te bekomen dat door iedereen kan begrepen worden die betrokken is bij antiseismische bouwkunde en dat op alle gebruikers van toepassing is.

Naast deze strikte activiteit in de voorbereiding van deze documenten, vond reeds een eerste seminarie plaats voor architecten, ingenieurs en bouwbedrijven in februari 2003. Eind 2003 zal er opnieuw een seminarie plaatsvinden.

E. Trefwoorden

seismiek – aardbeving – België – EUROCODE-8 – niet-geïngenieerde bakstenen structuren – lage seismiciteit – bodemeffecten.

1. INTRODUCTION

EUROCODE-8 is the reference standard for the « Seismic Design of Structures » at European level. To be applicable at national level, EUROCODE-8 needs to be accompanied by a National Application Document in which the country gives values of Nationally Determined Parameters concerning for example the specific seismic action valid for the country and adapts rules related to specific national materials and types of buildings. The objective to determine some of these specific values and rules is at the base of the present study.

The project has been divided in 3 main objectives:
- Determine correct seismic action definition (response spectrum) at bedrock level
- Determine correct seismic action definition (response spectrum) at surface level
- Determine design rules for standard « non engineered » buildings suitable for Belgian seismicity

The seismologists and geologists research teams have treated the two first objectives (Royal Observatory of Belgium and Laboratory of Engineering Geology, University of Liege). The civil engineer's research team (Mechanical and Structures, University of Liege) has taken charge of the third objective.

1.1. Definition of a correct seismic action

The definition of the seismic action to design engineering structures in the EUROCODE-8 framework has to be done by considering peak ground acceleration (PGA) levels with a 10% probability of exceedence during the next 50 years, which corresponds to a return period of 475 years. These values of PGA have been evaluated for the Belgian territory (see Annex 1) for the bedrock in the seismic hazard map established in 2000 by the Liege University in cooperation with the Royal Observatory of Belgium (Leynaud et al., 2000). The information provided on this map furnishes only partial information as it does not take into account two fundamental parameters of the effects of earthquakes at the ground surface:

- <u>The duration of the ground movement and its frequency content</u>. This is the reason why it is necessary in the « structural dynamic computations » to associate response spectra with the seismic hazard map. For a given site, those spectra are normalised with the PGA of the hazard map.

- <u>The possible site effects caused by soil conditions</u>. It has been known for a long time that the local geological structure of a site can amplify and modify the frequency content of ground movements produced by earthquakes. Observations and measurements conducted during the earthquakes of Liege in 1983 and of Roermond in 1992 indicate that peak acceleration greater than that indicated on the hazard map already occurred recently. These data show the necessity to not neglect those site effects in the evaluation of the earthquake impact on an engineering structure.

The EUROCODE-8 European norm (see Annex 2) proposes response spectra for different soil conditions (classes A, B, C, D and E) and for two types of earthquakes. Type 1 corresponds to a spectra to apply in active seismic regions with earthquakes of magnitude greater than 5.5. Type 2 has been defined for low seismicity countries, as Belgium, and corresponds to earthquakes with magnitude less than 5.5.

This research part has two objectives:

1. to provide scientific arguments to decide if the response spectra to use in Belgium are solely that of type 2 or if it is well advised to also consider type 1 spectra;

2. to verify that the EUROCODE-8 response spectra for the different soil categories are sufficient and applicable to the different soil types of Belgium, particularly in the regions where the soft sedimentary cover is important.

The conclusion of this work should be to propose improvement in the spectra defined in the EUROCODE-8 in order to make them applicable in Belgium.

The first objective requires the definition of (a) reference earthquake(s) for a return period of 475 years. To reach the second objective, it is necessary to study site effects for typical Belgian soil configurations which are not clearly defined in the EUROCODE-8 soil classes A, B, C, D or E and to control the suitability of the proposed EUROCODE-8 response spectra by comparison with that calculated taking into account the site effects.

1.2. Design of standard « non engineered » buildings suitable for Belgian seismicity

The third objective of the research is related to the resistance to earthquakes of usual non-engineered buildings. The EUROCODE-8 is particularly well detailed for the structures designed by engineers in zones of medium to high seismicity. For the regions of low seismicity (as Belgium and North Europe) and for the non-engineered buildings like houses, the building criteria in EUROCODE-8 are very qualitative. The design indications present in EUROCODE-8 for non-engineered structures state correct principles, but quantitative application of the rule is impossible. For instance, concerning the structural links between walls and floors in masonry structures, it is stated that: “walls and floors are connected in every direction”; but how strong the connections have to be and how they must be laid out remain a mystery. In a previous study (Plumier et al., 1999) realised at University of Liege, it has been shown qualitatively that the layout of walls and the standard construction details in use in Belgium for simple structures may be really unsafe and explain dangerous partial collapse of structures, fall of walls, openings of cracks,… of the type largely observed after the Liege earthquake in 1983. It results from that situation that actually the architects are trapped between two extreme attitudes: or design as usual and produce unsafe structures (3000 houses abandoned after the Liege earthquake) or realise uneconomical details, standard in high seismicity regions, like peripheral concrete tie beams, which are probably excessive in Northern Europe. Both attitudes are unsatisfactory, but in practice only the first one prevails, at the expenses of public safety.

The main objectives of this research part are:

- the evaluation of internal forces in a set of real existing design (houses) to have an order of magnitude of the connection forces between constructional elements (walls, floors, roofs); this evaluation will allow the definition of Prescribed Action Effects.
- the design of connecting details between constitutive parts, their layout in the various critical parts of the buildings and the production of design rules for end users concerning these simple structures; these design will be used to develop Design recommendations and technical guides.

The challenge is to obtain low cost details adapted to low seismicity regions, not reproducing solutions valid for high seismicity, adapted to real need and modifying little the existing constructional practice.

2. METHODOLOGY

2.1. Seismic ground motion in Belgium

To define the response spectra to use for the design of a construction at a particular site, it is necessary to consider the physical characteristics of the potential seismic source and also the geotechnical nature of the soil at the site. Our purpose is first to investigate if the proposed EUROCODE-8 response spectra to use in Belgium belong to type 2 (earthquake with $M < 5.5$) or if it is well advised to consider also that of type 1 (earthquake with $M > 5.5$). Secondly, there are in Belgium regional geological configurations (thick sedimentary cover in the north of the country), which are not clearly represented in the EUROCODE-8 soil definition. It is important to examine the applicability of the EUROCODE-8 response spectra compared to that calculated taking into account these particular site effects.

2.1.1. Reference earthquakes – Response spectra on the bedrock

In the framework of the EUROCODE-8, the seismic event to consider is that corresponding to a return period of 475 years. The source characteristics (magnitude and duration) of the reference earthquake(s) will be used to evaluate the strong ground motions duration and their frequency content. The knowledge of these characteristics of the ground motions allows associating response spectra to the seismic hazard map. For a particular site, these spectra are normalised by the peak ground acceleration (PGA) reported on the 475 years seismic hazard map on the bedrock.

The determination of these reference strong ground motions and the associated response spectra results from a two step procedure: (1) to define the reference earthquake(s) with their source characteristics, and (2) to deduce ground motions and response spectra from the source characteristics obtained at the first step.

(1) Characteristics of the reference earthquake(s)

We adopted a simple earthquake source model (Brune, 1970), characterized by two parameters:

1) the seismic moment which represents the source dimension

$$M_0 = \mu.A.u$$

μ is the focal zone shear modulus, A the affected fault surface and u the average slip in the fault plane. The magnitude M is in direct relationship with the seismic moment: $M = 2/3 \log M_0 - 6.7$. (Hanks and Kanamori, 1979).

2) the source duration which is in direct relationship with the duration of the rupture propagation in the fault plane. This duration is the inverse of the corner frequency observed in the displacement spectra of S-waves in the elastic far-field.

The magnitude (or equivalently the seismic moment) has been evaluated in two different ways:
-by studying the strongest historical earthquakes having occurred in our region;
-by extrapolating the magnitude-frequency distribution to a return period of 475 years.

The frequency characteristics of the source have been defined by one parameter, the source duration. It has been estimated on the basis of the scaling laws of the seismic spectra calculated for recent instrumental earthquakes having occurred in Belgium.

(2) Reference response spectra on the bedrock

From the source characteristics of the reference earthquakes, accelerograms and response spectra have been calculated for the surface of the bedrock. Three different methods have been used:

(a) the search in the available strong-motion databases of real accelerograms recorded on bedrock conditions and corresponding to earthquakes with characteristics similar to that of our reference event(s);
(b) the calculation of synthetic accelerograms by the stochastic method of Boore (Boore, 1996; Boore and Joyner, 1997);
(c) the use of spectral attenuation laws established by different authors from the available strong ground motion database.

2.1.2. Site effects typical for Belgium – Reference spectra for these soils

When soft deposits overlay a bedrock, the waves are amplified at the resonance frequency of ground layers (site effect). Every site can be described by this frequency (fundamental frequency) that varies according to the thickness of the soft layers and by its dynamic elastic characteristic (Jongmans, 1990). In the EUROCODE-8, site effects are taken into account by the specification of response spectra corresponding to different classes of soils. These soils are classified in function of their geotechnical properties up to a depth of 30 m. The geological structure of the North of Belgium, mainly constituted by soft sediments laying on Palaeozoic basement with a thickness of the soft formations between 0 m in the South and hundreds of meters in the North, corresponds to a soil configuration not considered in the EUROCODE-8. Thus, it is fundamental to verify the applicability of the defined response spectra for these soils characterized by a strong thickness of soft sediments. For doing that, we studied in detail four different sites located in Liege, Uccle, Gent and Mons to evaluate their site effects. By convolving the obtained time response of the site effect with accelerograms on the rock, we evaluated response spectra for these sites and we compared them with those proposed by the EUROCODE-8 for similar site characteristics in the upper 30 m of soils.

Two experimental approaches are now commonly used to evaluate site effects by the analysis microtremors measurements: the Horizontal over Vertical spectral ratio (called H/V) and the array methods.

H/V spectral ratio

During the last 30 years, various techniques have been studied to assess site effects: numerical models, the spectral ratio method and the H/V ratios method (Riepl et al., 1998). This last one, initially proposed by Nogoshi and Igarashi (1971), and updated by Nakamura (1989), uses one single station. It is very quick to implement and at a low cost.
Its principle consists in recording the ambient vibrations of the ground during a period of time and in calculating the spectral ratio of the horizontal component over the vertical component. As shown by numerous authors (e.g. (Lermo and Chavez-Garcia, 1993; Teves-Costa et al., 1996; Theodulidis and Bard, 1995), the resulting curve generally shows a peak at the resonance frequency of the site. The objective of this first part of the study is to assess the fundamental frequency in the Brabant Massif from ambient vibrations measurements.

Theoretical aspects.

The idea of looking at the single station spectral ratio between the horizontal and the vertical components was first introduced by Nogoshi and Igarashi (1971). They showed its

relationship to the ellipticity curve of Rayleigh wave, and took advantage of the coincidence between the lowest frequency maximum of this H/V curve with the fundamental resonance frequency, to use it as an indicator of the underground structure (Bard, 1998).
The interpretation is based on the assumption that noise predominantly consists of surface waves. Under that assumption many authors (among which Field and Jacob, 1993; Lachet and Bard, 1994; …) agree the following arguments:

- The H/V ratio is basically related to the ellipticity of Rayleigh waves, because of the predominance of Rayleigh waves in the vertical component.
- This ellipticity is frequency dependent and exhibits a sharp peak around the fundamental frequency for sites displaying a high enough impedance contrast between surface and the deep materials. This peak is related with the vanishing of the vertical component, corresponding to a reversal of the rotation sense of the fundamental Rayleigh wave, from counter clockwise at low frequency, to clockwise at intermediate frequency.

This technique has been updated by Nakamura (1989, 2000). Its assumptions are in contradiction with the preceding authors. In short, he considers that noise is composed of body waves and surface waves among which are Rayleigh waves. These ones alter the information brought by the reflected SH waves, which are, according to him, predominant to explain the presence of the resonance peak. In this sense, the spectral division is designed to get rid of the effects of Rayleigh waves in the horizontal components. In this way the resulting curve H/V highlights the resonance frequency of the sites and their corresponding level of amplification as well.

Implementation and data acquisition.

The recording of seismic noise was made with the equipments of the Royal Observatory of Belgium, constituted by one acquisition unit PCM-5800 build by Lennartz Electronic, one tape recorder and one LE-3D Lennartz sensor (5 seconds and 3 components). The adopted sampling rate is 250 Hz and the length of each recording was 20 minutes. Many tests have been conducted to evaluate the amplification of the seismic signal necessary to obtain a sufficient numerical resolution for our calculation.
The Royal Observatory of Belgium and the Laboratories of Engineering Geology and Geophysical Prospecting (University of Liege) worked together to achieve the noise measurement campaign over 47 sites located in the Brabant Massif. The figure 1 exhibits the location of all those points. Their distribution is as homogeneous as possible over the whole area.

Array processing

At the opposite from the method presented in the last paragraph, the array methods are based on the simultaneous recording of ground motions at various points (between 4 and 22 sensors in our case). The sensor arrays are an interesting way for obtaining the shear velocity profile at a given site, comparing to classical approaches (SH refraction, active source surface waves, cross-hole …). They present the following advantages:

- Easy implementation in whatever site (especially urban sites) due to the absence of active source.
- High investigated depth (from tens of meters to hundreds meters according to the inter-distance between stations and the frequency range)

Figure 1: Location of the sites where noise measurement have been conducted with the purpose of calculating the H/V ratio.

As for the H/V ratio (Nogoshi and Igarashi's point of view) the interpretation is based on the assumption that noise consists mainly of surface waves. This is true when the dynamic characteristics of the soil structure support the generation of this kind of waves, e.g. when soft sediments lay on a rigid bedrock (Jongmans, 1990; Scherbaum et al. 2002). In a layered model, surface waves are dispersive: their phase velocity is a function of the frequency according to a dispersion curve. In a 1D case it is straightforward and quick to compute the dispersion curve from the seismic velocities P (compression) and S (shear), the thickness and the densities of all layers constituting the ground model (Aki and Richard, 1983; Thomson, 1950; Haskell, 1953). Among the elastic characteristics, the shear velocity is the parameter having the most influence on the results.
Inside the propagation plane, compression (P) and radial shear (SV) waves interfere to produce Rayleigh surface waves. On the other hand, Love surface waves are polarized in the transverse direction (SH) and contain only shear waves. On the vertical component of the sensors we always measure Rayleigh waves whereas we observe both types of waves on the horizontal components, resulting in a more complex procedure to extract the velocity information (Ohrnberger, 2001). For this reason we limited our study to the vertical Rayleigh case.

In a noise recording, high frequencies are strongly influenced by the characteristics of the superficial layers while the phase velocities at low frequency are related to the deeper ground materials. The ratio between the aperture of the array and the wavelength of ambient noise must be correctly tuned in order to achieve a good resolution. If the distance between stations is greater than the wavelength the resulting curves will be aliased (false higher velocities). Fäh et al. (2001) limit the acceptable frequency band between the maximum and the minimum of the ellipticity of the fundamental Rayleigh mode. Scherbaum et al. (2002) note that the ground itself acts as high-pass filter reducing the available frequency bands. In the investigated sites, it has not been possible to find stable phase velocity below the resonance frequency measured by the H/V spectral method, in agreement with the preceding authors.
The interpretation of noise recording is generally divided in two distinct parts described hereafter (figure 2): (1) derivation of dispersion curve from signals and (2) inversion of the velocity profiles (Vs as function of depth).

Derivation of the dispersion curve
Various methods are currently available to extract the phase velocities from the measured time series:

- Beam Forming – FK method (Lacoss et al., 1969);
- High Resolution Beam Forming method (Capon, 1969);
- Spatial Autocorrelation method (Aki, 1957), modified by Bettig et al. (2001) for irregular arrays (MSPAC).

We used the first method that was available at the time we computed the results of this study. Under the assumption of stationary noise in space and time, it is possible to identify, for each frequency band, a preferential velocity and direction of propagation. As we assume that the ambient vibrations contains a large part of their energy as surface waves, we can obtain the phase velocities of Rayleigh or Love waves and we construct in this way the dispersion curve.
The signal processing is applied on a short portion of the whole recorded signal (time window). As a final step, some statistics over large number of small independent time windows are performed to give a median and standard deviation for dispersion curve (figure 2 after step 1).

Calculating a dispersion curve from the Vp, Vs, density and thickness of all layers is numerically possible and results in only one unique solution (forward problem). Calculating a model from a dispersion curve is much more difficult (inverse problem) as the same dispersion curve can be explained by various distinct models (non uniqueness of the solution) and as no analytical expression exists. Inverse methods are distributed in two main categories:

- Importance sampling methods that use random search of the parameter space (Monte Carlo type)
- Least-square methods that converge to the solution by means of the partial derivatives and a series of iterations for non-linear cases.

Very often, the software used for the inversion of dispersion curves is Herrmann's code (St Louis, 1986) based on a least-square method. This classical approach suffers some strong disadvantages:

- The result is only a single model supposed to be the optimum;
- The risk of inversion trapped in a local minima is sometimes important and depends on the starting model;
- Manual control of the damping factor prevent any automatic fit;

On the other hand, random search algorithms can determine a set of equivalent models having the same reproduction of the original dispersion curve. During the last ten years several of them were used in the geophysical domain:

- Genetic algorithms (Lomax et Snieder, 1994);
- Simulated Annealing (Sen and Stoffa, 1991);
- Neighbourhood Algorithm (Sambridge, 1999).

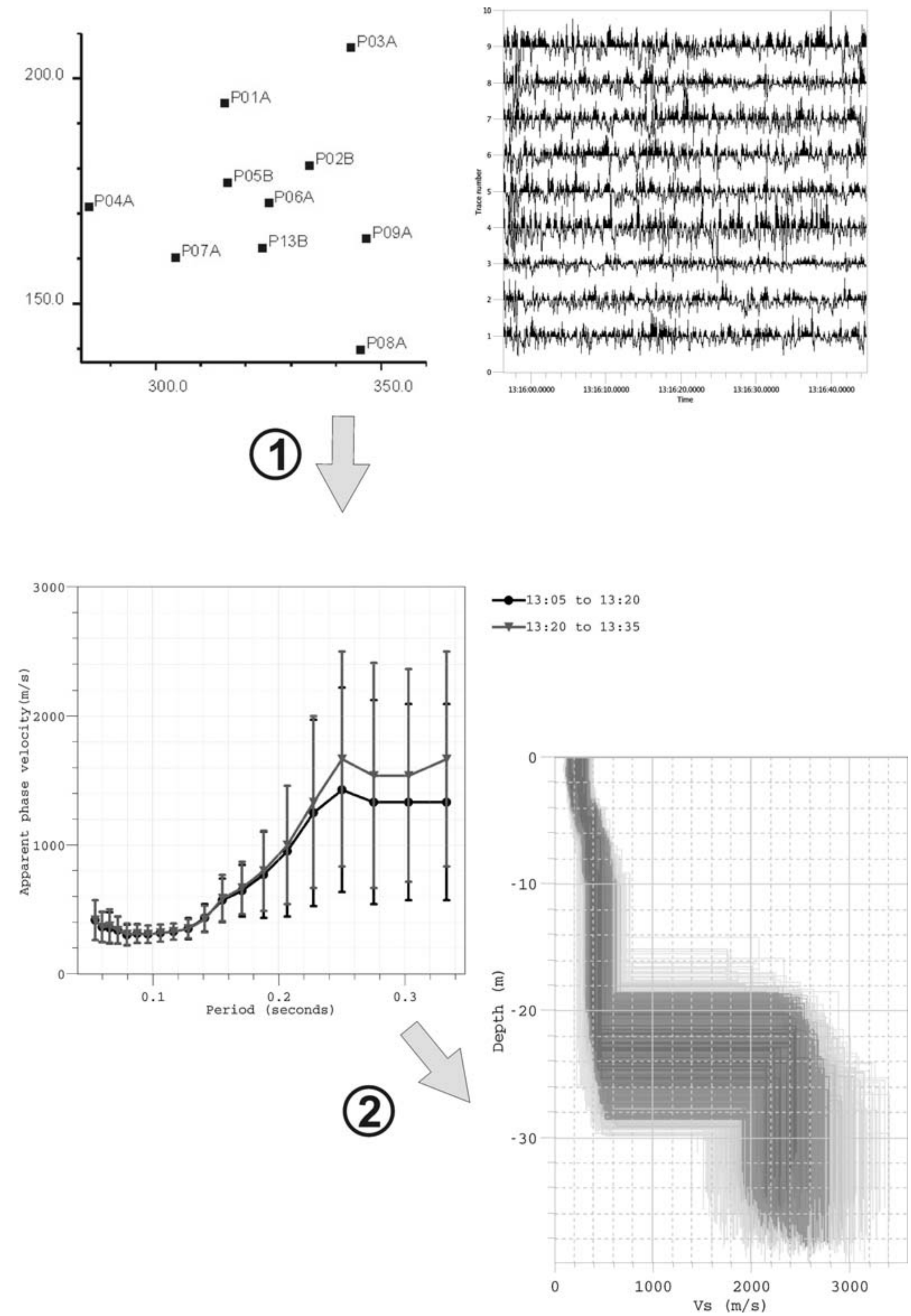


Figure 2: flow chart of array data processing used to extract the velocity information - Inversion of velocity profiles

During this project we have implemented the Neighbourhood Algorithm and we linked it to a revised version of the forward computation written by Herrmann (1987). FORTRAN codes were translated in C++ and inside loops have been speeded up resulting in a time consumption divided by 3 to 4. A robust automatic quality control has been added at the end to be sure that the calculated curve does not suffer from any mode jumping supposed to occur in many situations. The developed inversion code has been tested on several synthetic models with 2 or 3 layers where the Vp, as well as Vs, could be correctly deduced. For curve with strong standard deviations, as it is the case for noise measurements, the non uniqueness of the model is important and direct search algorithms have proved to be an efficient solution.

Response spectra on the studied sites

To calculate the response spectra on the four studied sites, the following procedure has been used:

1. A response spectrum on the bedrock has been calculated for each site from real and synthetic reference accelerograms (see 2.1.1). The spectral accelerations are scaled at high frequency with the PGA indicated on the seismic hazard map (on the rock);
2. Convolve the reference accelerograms recorded on the rock with the transfer function of the site effect to obtain the accelerograms corresponding to the site;
3. Calculate the response spectra from these accelerograms with a scaling factor equal to that obtained in 1.;

The resulting spectra characterize the studied sites for a return period of 475 years and can be compared with the EUROCODE-8 response spectra for the soil conditions corresponding to the sites up to a depth of 30 m.

2.2. Vulnerability in low seismic activity zones

The methodology used to study the structural resistance of non-engineered buildings is divided as follows:

- To select eight typical Belgian simple building structures
- To analyse them by 3D Finite Element modelling
- To deduce from these analyses the connection forces between constructional elements (walls, floors, roofs).
- To design and sketch the connection devices and their layout in the various critical parts of the buildings
- To produce design rules for end users concerning these simple structures.

The first goal of the study is to define the forces induced by the possible Belgian earthquake at the junctions between the elements that form the box constituting a non-engineered masonry dwelling.

Earthquakes induce forces in the structures because of their inertia. These forces result from the tendency of the buildings to stay at a fixed point (motionless), and so to resist to the movement. Similar effects are produced if the soil is stationary and a horizontal force is applied. For this reason, the effect of an earthquake presents similarities with the wind effect. However, wind effect is proportional with the front area of the structure while the seismic effect is proportional with the mass of the structure.
The conventionally non-engineered buildings resist horizontal forces (wind or earthquakes) owing to their box configuration (see Figure 3). This configuration offers resistance by its constitutive elements: roof, floors and walls.

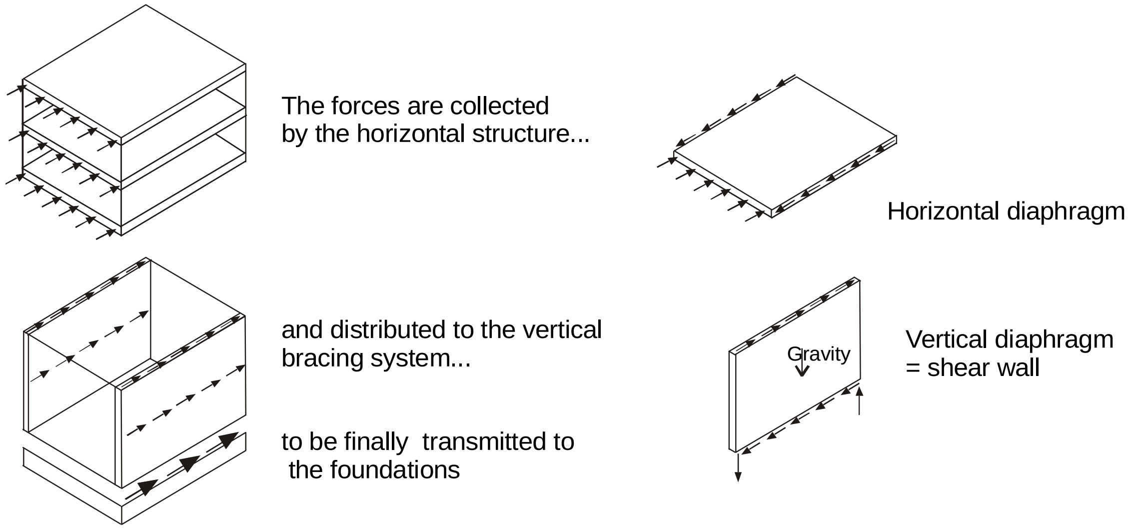


Figure 3. Box behaviour of a non-engineered structure.

The horizontal forces (wind or earthquakes) are transmitted to the foundation of the structure by the bracings system of the building.
The bracings system is composed of 2 types of elements:

- horizontal bracings, also called diaphragms, and represented by floors and roofs
- vertical bracings, represented by the walls.

The role of the horizontal bracing is to transmit the lateral actions to the vertical bracings, which transmit them to the foundations. To ensure its role the horizontal bracing has to be linked to the vertical structure. It is necessary to have horizontal bracings at every level of the building, including the roof level.

The efficiency of diaphragms (horizontal bracing) is also related to their stiffness. If they are rigid (concrete floors for example), that means if they may resist shear and bending in their plane, they ensure an equal displacement of the vertical elements that they connect, essentially the vertical elements parallel to the applied action.
Flexible diaphragms (timber floors) are a priori not so efficient, especially to transmit global torsion effects. Since they are present in a lot of old Belgian houses, they will also be studied to find solutions to improve their behaviour for seismic resistance.

In Plumier et al. (1999), it has been shown that most of the damage to the houses due to the Liege earthquake of 1983 was related to the following factors:

- the tendency to fall over of the walls perpendicular to the seismic actions;
- the lack of connection between walls, between walls and floors, and between walls and roof;
- the lack of walls in the direction parallel to the earthquake

The walls parallel to the applied action resist best to horizontal forces and work as shear walls (D'Ayala et al., 1997). If these walls are correctly built and anchored (dimensions, design and construction), if they are present on the four sides of the building, if roof and floors work as diaphragms and if the wall-to-wall links, the wall-to-floor links and the wall-to-roof links are resistant, the whole building will work as a box. In this case, the forces induced by low to moderate earthquakes should be supported with low damage. That is what has to be controlled in the present study, not on theoretical houses, but on real buildings.

As each real structure represents a particular case in terms of dimensions, position of the rooms, openings of the doors and windows, stairs, roof, … the forces that we are searching for have values and distributions linked with each structure. It does not exist a precise mathematical value of these forces between walls, between floors and walls, and between roofs and walls, which would be independent of the considered building.
Consequently, the way of determining these forces has to be statistical, based on the examination of a set of real buildings.

So, eight houses were selected, with different wall configurations in plan, different floor types (monolithic concrete slab, prefab elements slab, timber floors like in traditional housing and in post modern housing) and different roof types (flat concrete slab, wooden frames). These eight houses are representative of the way houses are generally built in Belgium. The characteristics of the selected dwellings of the sample are given at Figure 4.

Numerical three-dimensional modelling of the selected structures submitted to gravity loads and earthquake action are done in order to compute the action effects in all the elements of the structure and at the junctions between the elements.

The numerical modelling has been realised with the Finite Element Program SAP2000 (2000). The main choices concerned the way of modelling

- masonry
- roofs
- seismic action.

Modelling the masonry

A lot of works found in the literature are focused on the understanding and modelling of the behaviour of a single shear wall (Lourenço and Rots, 1997, Lourenço et al., 1998, Lofti and Shing, 1991, Gamboretta and Lagomarsino, 1997a, 1997b, Pande et al., 1997, …). The modellings are then highly non linear and try to reproduce with great accuracy the behaviour of the wall in functions of different parameters such as the acting normal stress, the aspect ratio, the materials properties, the boundary conditions, the presence of one opening. The effort is great to obtain the behaviour of one single element.

Other authors try to model whole masonry buildings by using simplified macro models.
Brencich and Lagomarsino (1997) use macro elements made of rigid blocks, piers and architraves. Braga et al. (1997) discretise the masonry buildings through panel elements, linked to each other and to floors by particular constraint elements. Magenes (1995, 1997) models piers and spandrels by macroelements joined by rigid elements. These methods apply relatively easily to regular buildings, but are not applicable to non regular dwellings where the position and dimensions of the windows are not regular and where the slope of the roof gives triangular pieces of wall. Another limitation is the difficulty to use these models in three-dimensional configurations.

With the objective to obtain an order of magnitude of the forces between the critical elements of the dwellings (walls, floors, roof), it seemed that all the methods found in the literature were too complicated or/and in a too early stage of development. So it was decided to work with elastic models. The assumption is that the house forms a relatively good box, that the walls are cracked, but not in a critical stage (no expected failure) and that the forces between the walls, floors and roof give a good idea of the real forces when the structure works as a box.

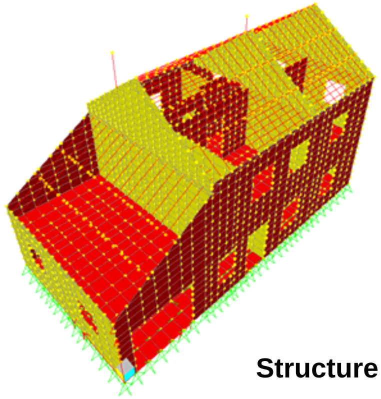
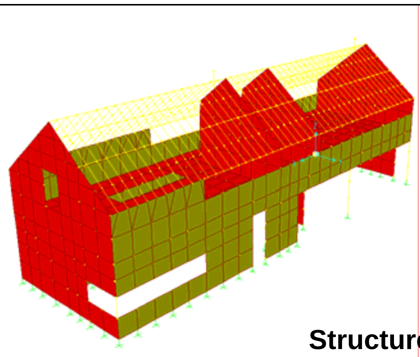
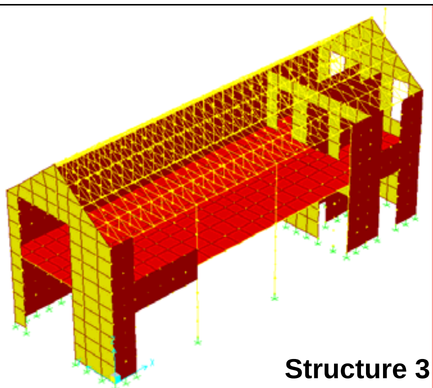
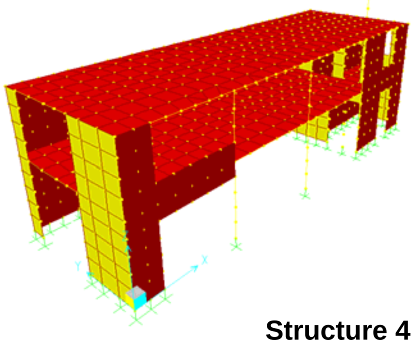
 Structure 1	**Structure 1**: new building max height: 5,85 m Length1 x Length 2: 13,7 x 6,6 m² walls: concrete blocks (19 cm) + exterior bricks (9 cm) prefabricated concrete slabs + timber floors wooden traditional roof – slope: 35° Percentage of openings by direction: 19 and 21 % Percentage of shear walls by direction: 3,1 and 3,9 % Range of floor span: 4 – 5,5 m Total mass: 169 tons Base shear resultant / total mass: 15 % g in each direction
 Structure 2	**Structure 2** : new building max height: 4,5 m Length1 x Length 2: 19,1 x 6 m² walls: bricks (19 cm) + exterior bricks (9 cm) prefabricated concrete slabs wooden traditional roof – slope: 35° Percentage of openings by direction: 30 and 22 % Percentage of shear walls by direction: 3,6 and 2,2 % Range of floor span : 6 m Total mass : 162 tons Base shear resultant / total mass: 5,3 % g , 16,5 % g
 Structure 3	**Structure 3** : new building max height: 5,55 m Length1 x Length 2: 16,1 x 5,4 m² walls : bricks (19 cm) + exterior bricks (9 cm) prefabricated concrete slabs wooden frames for the roof – slope: 40° Percentage of openings by direction: 36 and 28 % Percentage of shear walls by direction: 3,3 and 1,6 % Range of floor span: 5,4 m Total mass: 133 tons Base shear resultant / total mass: 8,7 % g and 16,4 % g
 Structure 4	**Structure 4** : new building max height: 5,55 m Length1 x Length 2: 16,1 x 5,4 m² walls : bricks (19 cm) + exterior bricks (9 cm) prefabricated concrete slabs prefabricated concrete roof – slope: 0° Percentage of openings by direction: 36 and 28 % Percentage of shear walls by direction: 3,3 and 1,6 % Range of floor span: 5,4 m Total mass: 153 tons Base shear resultant/total mass: 15,8 % g and 20,7 % g

Figure 4. Main characteristics of the studied houses

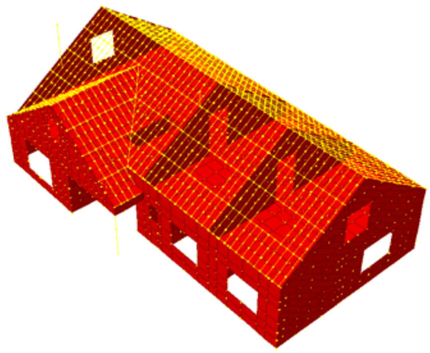
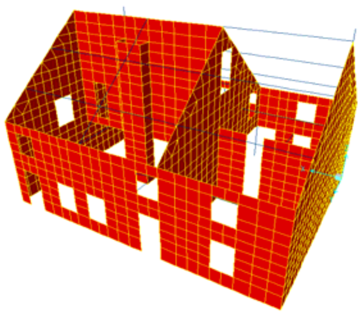
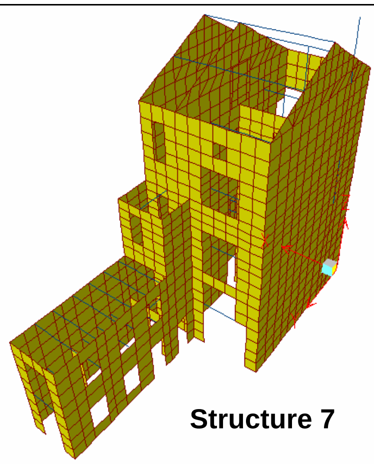
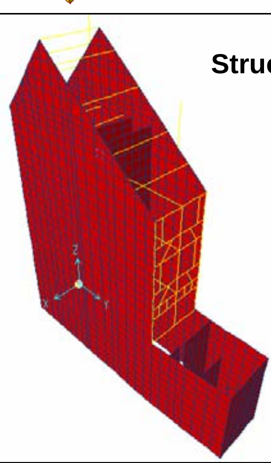
 **Structure 5**	**Structure 5** : new building max height: 3,5 m Length1 x Length 2: 16,65 x 10 m² walls: concrete blocks (14 cm) + exterior bricks (9 cm) prefabricated concrete slabs wooden traditional roof – slope: 35° Percentage of openings by direction: 28 and 13 % Percentage of shear walls by direction: 1,6 and 3,1 % Range of floor span: 4 - 5 m Total mass: 190 tons Base shear resultant / total mass: 11,8 % g and 7 % g
 **Structure 6**	**Structure 6** : old country building max height: 5 m Length1 x Length 2: 13,5 x 10,3 m² walls: natural stone (50 cm) and small bricks (40 cm) timber floors wooden traditional roof – slope: 45° Percentage of openings by direction: 26 and 5,7 % Percentage of shear walls by direction: 6,6 and 9,8 % Range of floor span: 5 - 8 m Total mass: 433 tons Base shear resultant / total mass: 13,7 % g and 7,8 % g
 **Structure 7**	**Structure 7** : old town building max height: 13,5 m Length1 x Length 2: 7 x 10 m² + 2,8 x 9 m² walls: small bricks (30 cm) timber floors wooden traditional roof – slope: 30° Percentage of openings by direction: 40 and 5,6 % Percentage of shear walls by direction: 1,8 and 12,6 % Range of floor span: 2 - 5 m Total mass: 455 tons Base shear resultant / total mass: 15,4 % g and 10 % g
Structure 8 	**Structure 8** : old town building max height: 19,2 m Length1 x Length 2: 16,3 x 3,7 m² gable walls: small bricks (40 cm) façade 1: stone beams with bricks infills façade 2: half-timbered wall timber floors wooden traditional roof – slope: 40° Percentage of openings by direction: 68 and 0 % Percentage of shear walls by direction: 2,4 and 22 % Range of floor span: 4 m Total mass: 570 tons Base shear resultant / total mass: 19,2 % g and 14 % g

Figure 4 continued. Main characteristics of the studied houses

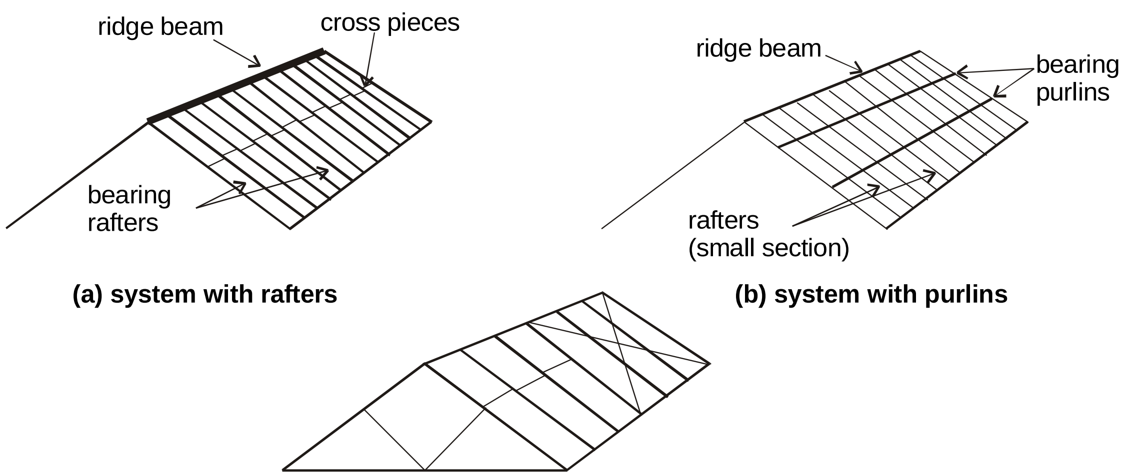
The masonry walls are modelled by means of shell elements. Their Young's modulus is calculated on the basis of Eurocode 6 and the cracked modulus on the basis of EUROCODE-8:

$$E_{cracked}(EC8) = 0,5\ E_{masonry}(EC6)$$

The modelling choices have been tested on small box structures. Some spurious membrane effects in the walls appeared due to the modelling with shells and are suppressed by taking a Poison's ratio equal to zero for the masonry walls.

<u>Modelling the roof</u>

The types of roofs considered in the study are given at Figure 5.



(c) system with roof trusses

Figure 5. Types of roofs considered in the study.

The greater the stiffness in plan of the roof is, the more important the forces between the roof beams and the gable wall are. This result is explained by the fact that the dynamic forces are function of the stiffness of the considered elements : a more flexible solid is characterised by a greater period of vibration and it results in smaller design accelerations if T > 0,5 sec (see the shape of the spectrum at Figure 6).

The stiffness of a roof is brought by all its components : main elements (beams, purlins, rafters), « under-roofing », roofing (tiles, slates),…

The roofs of the selected houses were modelled by their main timber elements. To take into account the non structural elements and to avoid the underestimation of the stiffness, some additional stiffness in plan was added by increasing the value of the inertia of the beam for the bending in plan.

On the other hand, a study of the sensibility of the roof-to-wall forces to the stiffness of the roof was made on a small model.

<u>Modelling the seismic action.</u>

Two types of analysis are possible : response spectrum analysis and time-history analysis under accelerograms. It was decided that to obtain an order of magnitude of the maximal forces that may occur during an earthquake, response spectrum analyses were the most efficient way to have an upper bound of the forces that may occur in the structure.

The design response spectrum is chosen as the envelope of the EUROCODE-8 spectra of type 1 and type 2. This choice was a secure choice. When the analysis started, no result existed to show whether the design spectrum of type 2 was representative of the Belgian earthquake action. The behaviour factor q is taken equal to 1,5 (typical value for unreinforced masonry structures).
The houses are supposed to be located in the Belgian zone 2 (acceleration at bedrock a_g = 0,1g) and on a EUROCODE-8 soil of class E (soil amplification factor S = 1,6 for type 2 spectra).
The corresponding accelerations at the foundation of the structures are a_gS = 0,16 g and the maximal spectral accelerations are 0,27 g.
The corresponding design spectrum is given at Figure 6. The range of the main vibration periods of the studied structures is also reported. It can be seen that the periods of vibration correspond mainly to the plateau values of the design response spectrum.

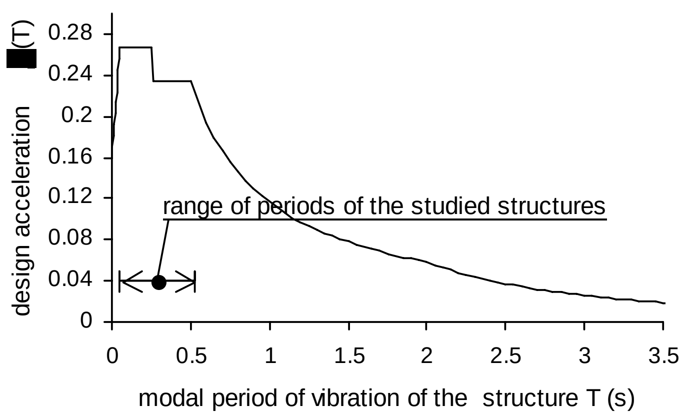


Figure 6. Design spectrum acceleration used in the numerical study. Envelope of type 1 and type 2 earthquakes. Belgian zone 2 (a_g = 0,1g). Soil E (amplification factor S = 1,6)

From the forces obtained by the numerical modelling, the required connecting devices between the elements (links of horizontal to vertical elements, links of vertical to vertical elements, etc…) for the different types of buildings studied are defined. Different technological possibilities of connections are considered (dowels, flat straps nailed, angles, reinforcement grids in most stressed zones, etc…).

3. RESULTS

3.1. Reference earthquake(s) – Response Spectra on the Bedrock

3.1.1. Characteristics of the reference earthquake(s)

To evaluate the source characteristics of the reference earthquake(s) to take into account in our evaluation of the EUROCODE-8 response spectra, it was necessary to re-evaluate the long-term seismic activity in our regions. Thus, we took the opportunity of this project to improve the catalogue of Belgian earthquakes and the knowledge on the local effects of some individual earthquakes. We re-evaluated in the new EMS-98 intensity scale the macroseismic information available for the 30 felt and destructive earthquakes in Belgium having occurred during the 20th century and for which an official inquiry was conducted by the Royal Observatory of Belgium. We assess also the magnitude of the strongest historical earthquakes having occurred since 1350 by comparison of the macroseismic data with post-1910 earthquakes for which a magnitude was determined by the seismogram interpretation.
As two different response spectra have been defined in the EUROCODE-8, our present discussion concerns the scientific arguments, in relationship to the earthquake source dimension, which should allow to decide if the spectra to use in Belgium are only that of type 2 (earthquakes with $M < 5.5$) or if it is well advised to consider also that of type 2 (earthquakes with $M > 5.5$).

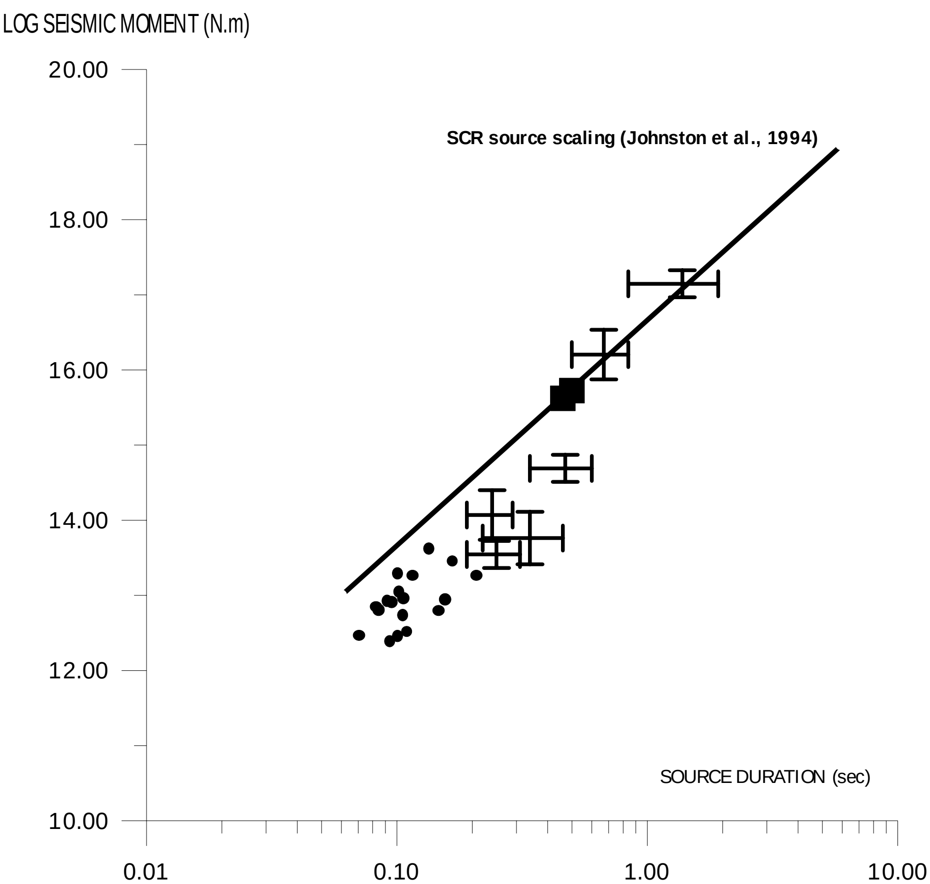


Figure 7. Relationship between the seismic moment and the duration of earthquakes in northwest Europe. The Johnston et al. (1994) relationship for stable continental region is indicated.

To determine the frequency content of the reference earthquake(s), we first deduced the scaling laws of seismic spectra for recent instrumental earthquakes and secondly, we extrapolated the obtained results at the magnitude of the reference earthquake(s). Figure 7 presents all the available information concerning the source duration of earthquakes having occurred in our regions (Ahorner, 1983; 1985; Camelbeeck, 1985; Camelbeeck and van Eck, 1994; Oncescu et al., 1994). For the largest earthquakes considered in this study, with a seismic moment greater than 10^{15} N.m (M greater than 4.0), the result correspond very well to the average relationship established by Jonhnson et al. (1994) for stable continental regions.
We evaluate the magnitude range having the strongest impact on the seismic hazard map for the Roer graben. The figure 8 indicates the calculated hazard (PGA) calculated in considering only earthquakes within a magnitude range of 1.2 centred on the indicated magnitude values. For a return period of 475 years, the hazard is maximal for the magnitude 4.4-5.6 window. The Peak Ground Acceleration diminish strongly for the window centred on M=5.3. For a return period of 10,000 years, the contribution of the large earthquakes greater than 6.0 becomes fundamental.

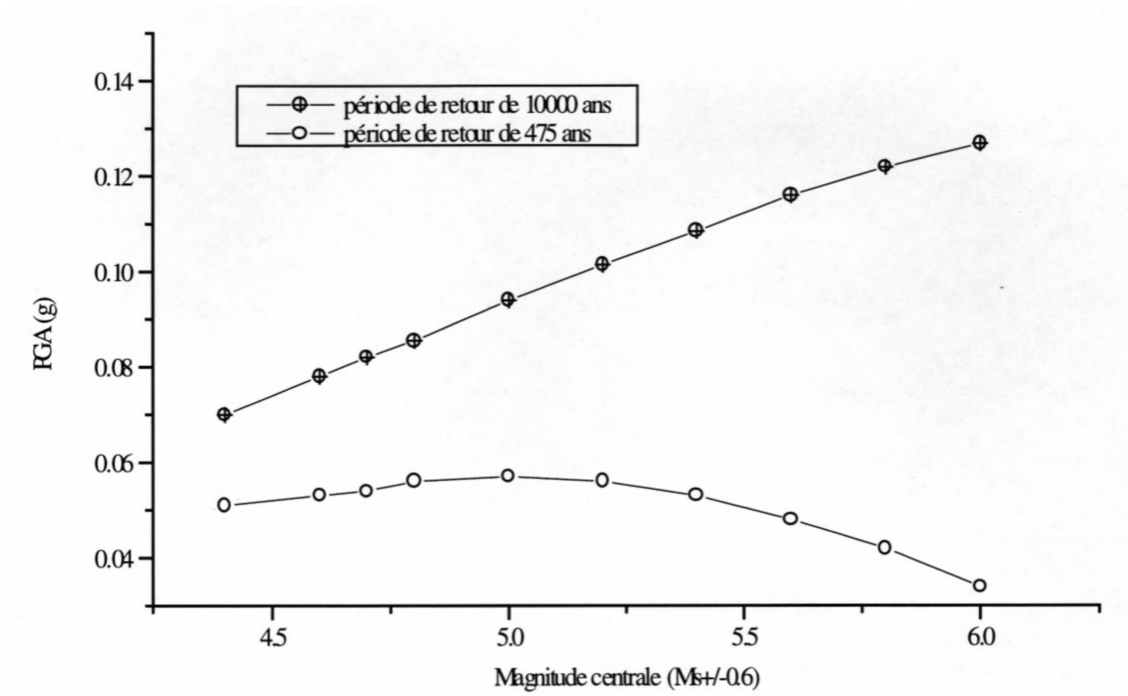


Figure 8. Seismic hazard in the Roer graben – Contribution of seismicity in different magnitude range (windows of 1.2 in magnitude). The hazard is calculated for sources at a depth of 15 km and return periods of 475 and 10,000 years.

On the other hand, the extrapolation of the annual earthquake frequency with magnitude (Figure 9) indicates that one earthquake of magnitude M=6.2 occur in average each 475 years. Historical seismicity data inform us that since 1350, three large earthquakes with a magnitude greater than 6.0 occurred in our region. Considering the fact that such an earthquake caused damages in a very widespread area, generally including all the Belgian territory, it should perhaps be well advised to consider it for the response spectra definition. Without taking a decision on the opportunity to consider type 1 earthquake in the application of EUROCODE-8 in Belgium, we defined two reference earthquakes:

- An earthquake with M=5.5 which gives a significant contribution to the 475 years return period seismic hazard.
- An earthquake around M=6.5, relatively rare, but which could occur in the region with a return period of 475 years.

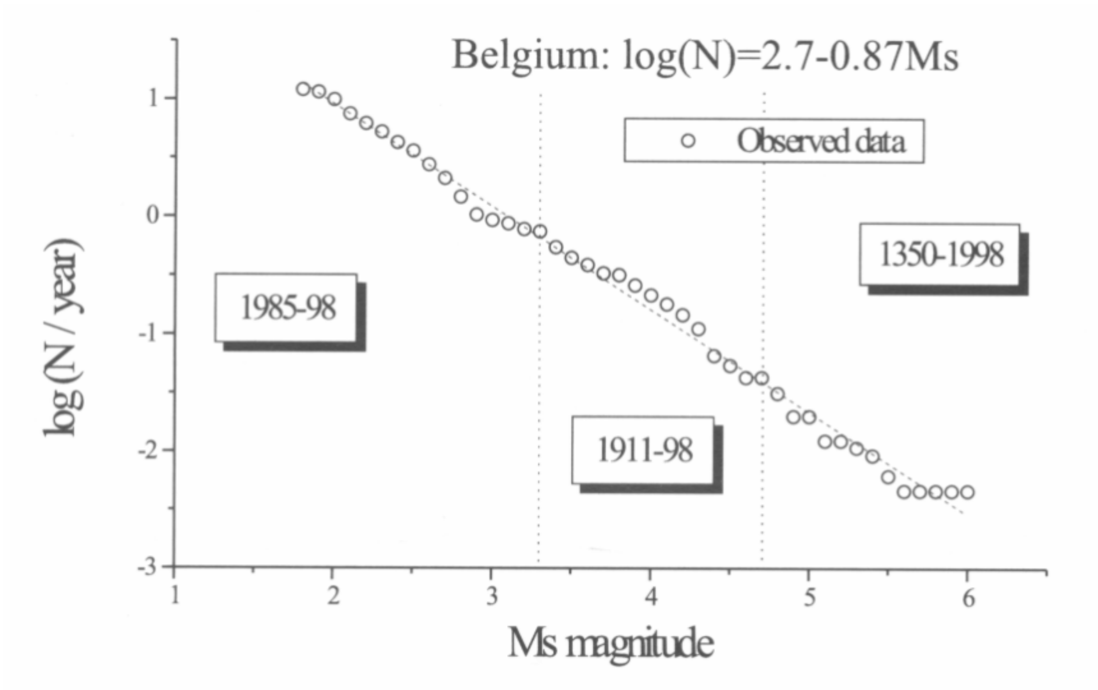


Figure 9. Cumulated annual earthquake frequency with magnitude in Belgium

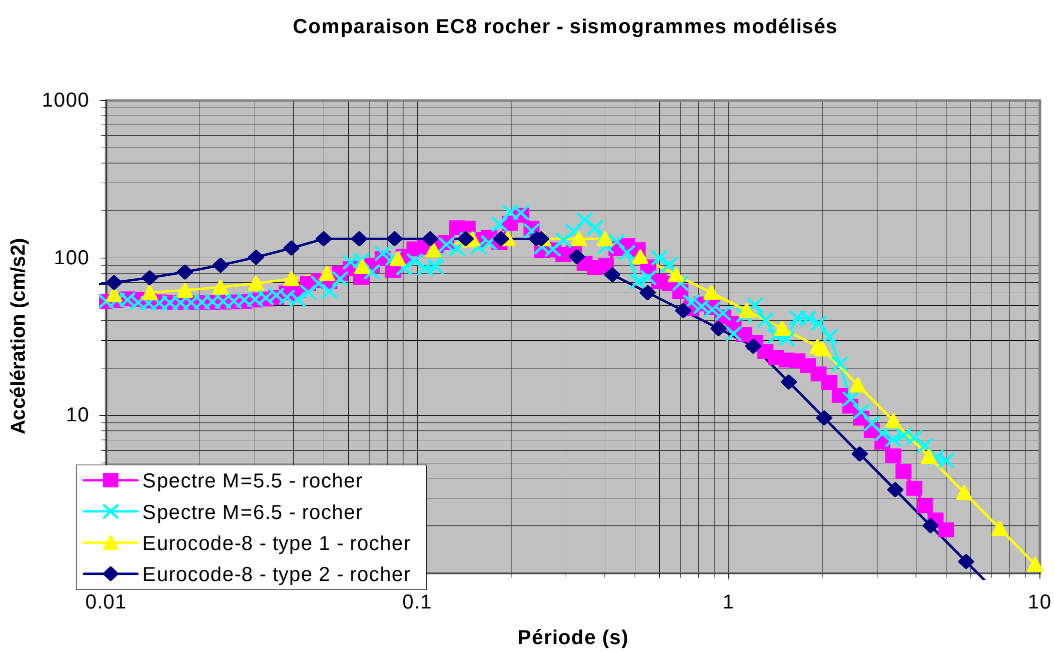


Figure 10. Comparison between the EUROCODE-8 response spectra on the bedrock and that calculated for our reference earthquakes.

3.1.2. Elastic response spectra on the bedrock

From the defined source characteristics for the reference earthquakes, accelerograms and the corresponding elastic response spectra on the bedrock have been deduced. The accelerograms have been determined in two ways: (1) by searching in available strong-motion data base time histories corresponding to stations on the bedrock for earthquakes having similar characteristics to that of our reference earthquakes; and (2) by calculating synthetic accelerograms by the stochastic method of Boore.

The resulting response spectra are compared with that of the EUROCODE-8 on the bedrock (Figure 10). Figure 10 presents the spectra corresponding to two earthquakes of respective magnitude 5.5 and 6.5, recorded at a distance of 5 km for which the time history has been calculated by the stochastic method. Their spectra are adjusted by an arbitrary value of PGA to the EUROCODE spectra corresponding to the types 1 and 2. To be more rigorous in our approach, we should have to calculate a large number of different time histories by the stochastic method in considering the uncertainties in the source parameters and then to consider the average response spectrum and the associated variance. Unfortunately, the quantity of work to conduct and the limited duration of the project don't allow realizing this statistical approach. This remark is equally valid for the response spectra calculated in taking into account the site effects.

The EUROCODE spectra on the bedrock appear well representative of the spectral content of the modeled strong ground motions for our two reference earthquakes.

3.2. Site Effects typical for Belgium – Reference Spectra for these Soils

3.2.1. Site effects in the northern part of Belgium – Use of H/V spectral ratio

Forty-seven microtremor measurements (see Figure 11) were performed throughout the Brabant Massif and the Campine basin. We used LE-3D/5-sec seismometers connected to the classical PCM-MARS 5800 station build by Lennartz Electronic. These acquisition systems contain a 12 bits A/D converter. The Least Significant Bit corresponds to 1 μV and the sensitivity of the seismometer is 400 Vs/m. To be confident in the reliability of our measurements, tests have been conducted on the electronic noise caused by the electronic devices of the seismometer. Without amplification of the input signal, the electronic noise appear important in the frequency band 0,1 – 0,4 Hz. Tests were satisfactory with an amplification of 2^4. All the measurements have been done with such an amplification. In this way, we benefited of the maximal possible digital resolution. The measurements have been conducted in order to avoid different possible sources of perturbation. The seismometer was generally protected against the wind although its influence can hardly be removed completely. Table I contains the sites name and number, their latitude and longitude as well as the measured frequency. The microtremors were recorded during 20 minutes at each site with a frequency sampling of 250 Hz. H/V ratios were computed on twenty 120-sec windows with an overlap of 60 seconds. The mean curve and the mean $\pm$ one standard deviation curves were calculated at each site, except for Villers-la-ville where only one data set was available. Figure 11 shows the H/V ratio curves at twelve stations. Each column on this figure represents an approximate SN line. All curves exhibit a peak at a frequency that is usually very stable in time. Also, curve shape variations from a sharp peak to a broad amplitude range are observed. This modification could result from different measurement conditions, including the weather influence. However, the peak frequency remains clear and stable for all stations. The main feature on Figure 11 is that, except between stations Haneffe and Broekom, a systematic decrease of the resonance frequency is observed from South to North, from a few Hz in the southern part to about 0,3 Hz in the North. The regular increase of the fundamental period observed from South to North is in good agreement with the augmentation of the soil thickness. The discrepancies observed in the vicinity of Haneffe concern three sites where the bedrock is shallow, could be due to the presence of soft superficial layers in the area but this hypothesis has to be verified. Figure 12 presents the fundamental period as a function of the soil thickness for all the measurements and shows a gross correlation between the two variables. Supposing that the post Palaeozoic sediments are homogeneous, the natural period T of the layer is given by T= 4H/Vs where H is the soil thickness and Vs the shear wave velocity. The non-linear shape of the curve fitted to the data however indicates that Vs increases with depth (from about 120 m/s to 950 m/s) and that the sediments can not be considered as one homogeneous layer, which is consistent with the geological data and the evolution of compactness with depth. In our study, H/V spectral ratio measurements then appear to be a very quick and reliable way for mapping the sediment thickness overlying the bedrock at a regional scale and for pointing out anomalies resulting from variations in the sedimentary pile.

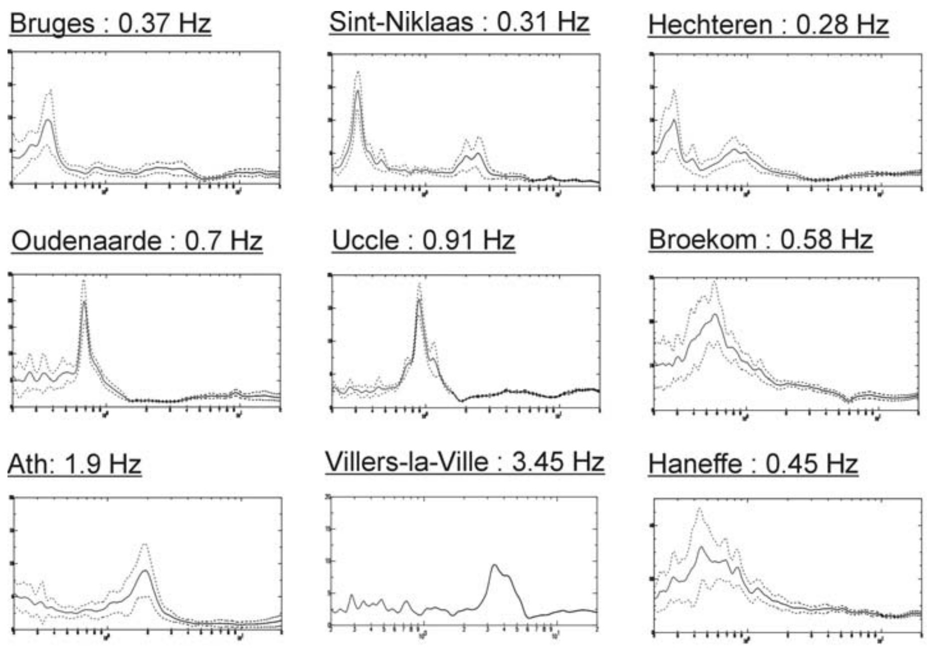


Figure 11. H/V spectral ratios determined at twelve stations in northern Belgium. The figure layout corresponds to the SN and EW directions.

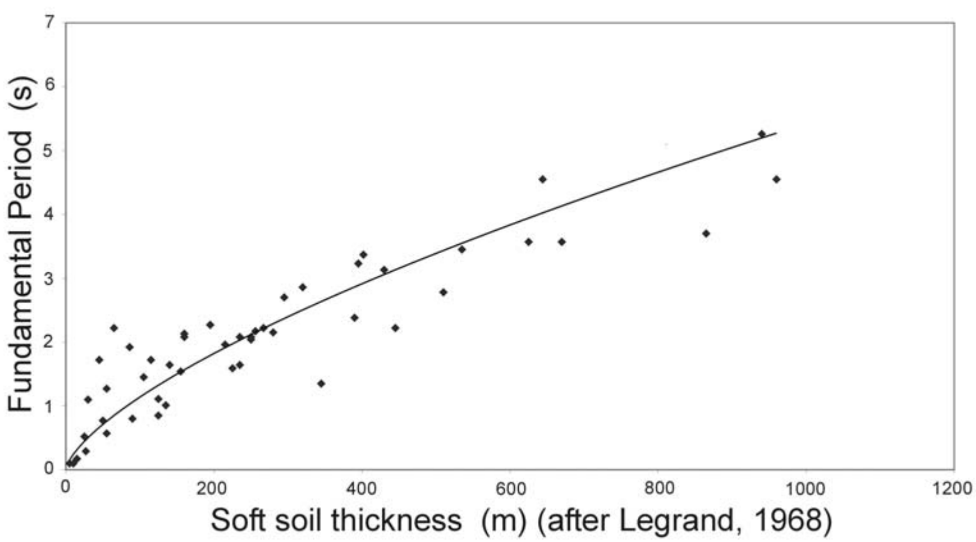


Figure 12. Relationship between the soil thickness and the fundamental periods derived from H/V method for all the studied sites in northern Belgium.

Table I. Location of and fundamental soil period at the sites where H/V measurements were conducted

Station number	Town	Latitude North	Longitude East	Measured frequency (Hertz)
1	De Panne	51° 05.384'	2° 34.955'	0.465
2	Nieuwpoort	51° 07.285'	2° 44.174'	0.49
3	Hoogveld	51° 03.704'	2° 54.957'	0.48
4	Knokke	51° 19.140'	3° 17.439'	0.32
5	Brugge	51° 11.875'	3° 18.557'	0.37
6	Eeklo	51° 11.908'	3° 36.060'	0.35
7	Assenede	51° 14.454'	3° 43.961'	0.297
8	ST-Niklaas	51° 12.454'	4° 4.817'	0.31
9	Doel	51° 18' 39"	4° 15' 36"	0.22
10	Mortsel (Antwerpen)	51° 9' 58"	4° 27' 44"	0.29
11	Westmalle	51° 16' 08.6	4° 41' 11.9"	0.22
12	Mol	51° 11' 2"	4° 39' 15"	0.27
13	Lommel	51° 14' 59.8"	5° 22' 37.2"	0.19
14	Kinrooi	51° 09' 09.1"	5° 45' 50.6"	0.17
15	Opoeteren	51° 4' 10"	4° 39' 15"	0.28
16	Hechteren	51° 04' 31.9"	5° 23' 46.8"	0.28
17	Hulshout	51° 04' 54"	4° 47' 07.2"	0.36
18	Mechelen	51° 1' 41"	4° 28' 49"	0.45
19	Gent	51° 3' 20"	3° 43' 42"	0.46
20	Ruiselede	51° 01.638'	003°24.362'	0.44
21	Tielt	51° 00.055'	003°17.181'	0.48
22	Ieper	50° 50.505'	003° 01.056'	0.61
23	Harelbeke	50° 52.515'	003° 18.394'	0.61
24	Dendermonde	51° 00.215'	004° 03.083'	0.51
25	Vilvorde	50° 55' 32"	4° 25' 15"	0.65
26	Wijgmaal	50° 55' 30.9"	4° 41' 34.1"	0.63
27	Diest	50° 59' 01.3"	5° 00' 43.7"	0.45
28	Maasmechelen	50° 59' 05"	5° 43' 36.1"	0.42
29	Hasselt	50° 54' 33.3"	5° 21' 44.2"	0.74
30	Oplinter	50° 50' 32.6"	4° 58' 27.7"	0.99
31	Tourines	50° 47' 07.1"	4° 45' 26.1"	1.18
32	Uccle	50° 48' 12"	4° 124 54"	0.91
33	Ninove	50° 49.332'	004° 06.062'	0.79
34	Oudenaarde	50° 49.703'	003° 37.331	0.69
35	Celles	50°42.727'	003°28.496'	1.74
36	Ath	50° 37.939'	003° 44.221'	1.92
37	Quenast	50° 39.778	004° 08.980	9.9
38	La Hulpe	50° 43' 51"	4° 29' 15"	1.25
39	Limelette	50° 40' 48"	4° 34' 22"	1.3
40	Orp	50° 42' 46.5"	4° 59' 29.8"	0.58
41	Broekom	50°46' 51.7"	5°19' 30.6"	0.58
42	Eben	50° 47' 15.5"	5° 39'	0.52
43	Haneffe	50° 37' 39"	5° 19' 50"	0.45
44	Liernu	50° 35' 30.8"	4° 48' 23.9"	0.91
45	Villers-la-Ville	50° 34' 39"	4° 31' 50"	3.45
46	Mellet	50° 30' 19"	4° 28' 44"	10
47	Tihange	50° 31' 51"	5° 15' 53"	0.59

3.2.2. Site structure in Brussels, Gent, Liege and Mons – array method

In addition to the 47 local H/V measurements conducted to evaluate the natural soil frequency, four urban sites in Liege, Brussels, Gent and Mons were studied in detail. The two main objectives of these local investigations were to determine the site transfer function in each of these sites and to evaluate the capabilities of seismic noise measurements using array methodology. This paragraph summarizes the results obtained at the end of each of the two stages described in the methodology chapter.

To validate the resulting velocity profiles we compared them to the reference logs defined for each site. These latter ones do not however to get information below 30 meters, although three sites (Brussels, Gent and Mons) exhibit a thick sedimentary covering of a few hundred meters. The passive experiments presented here aim at completing this lack of information for computing site effects.

<u>Site 1: Liège</u>

<u>Available data</u>

We chose the place of the ancient Bavière hospital located in the Meuse alluvial plain to deploy several arrays of distinct apertures. A good set of boreholes and cone penetration test data is available in the surroundings and on the site itself. The figure 13 displays a map with specifying the position of all sensors.

In all the boreholes the top of the bedrock is found at a depth of about 13 to 14 meters. The sediments are made of 3 to 5 meters of filling materials, 1 to 3 meters of clayey silts and 7 meters of sands and gravels. The depth of the basement is almost constant (between 10,5 to 13 meters) in the surroundings of the site.

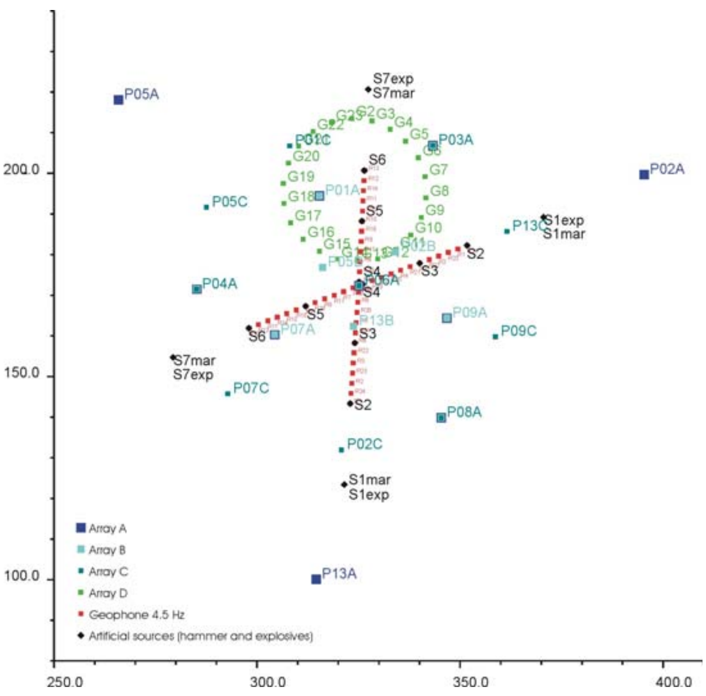


Figure 13: Site of Bavière in Liège, geophysical data

As shown on figure 13, two refractions surveys complete the noise recordings. The inverted seismic sections show a quasi 1D geometry with Vp around 500 m/s over the first 6 meters, increasing rapidly to 2000-3000 m/s at a depth of 16 meter. In collaboration with the Institute für Geowissenschaften der Universität (Potsdam), 3 arrays of 10 Lennartz 5 seconds have recorded the ambient vibrations for at least 1 hour each. The first layout (array A) has 1 sensor at the centre and 3 triangles imbricated and rotated by about 120 degrees, locating the stations at 25, 40 and 75 meters from the centre. The second one (array B) has the same geometry with different radii: 10, 25 and 40 meters. The third one (array C) kept the central station but all the others are distributed on a circle with radius equal to 40 meters. In reality, the geometry is not as perfect as described here but FK and MSPAC methods can adapt themselves to irregular arrays. A last array (D) composed of 22 usual refraction geophones (4,5 Hz for the resonance frequency) on a perfect circle (18 m of radius) terminates the list of available recordings. The acquisition system was a Geode from Geometrics where the maximum length of recording time per file was about 4 minutes (250 Hz for the sampling rate).

Dispersion curve

For arrays A to C, at least two independent time windows of 15 minutes were processed using FK method. For array D, we only used 4 minutes windows. The figure 14 shows all the dispersion curves obtained for each section and for each array.

The consistency and stationarity of the results are usually very good for all periods below a certain threshold, varying from one array to another. This limit is related to the resonance frequency measured by the H/V method and to the array's aperture as well. At high frequencies (lower periods) large aperture arrays show a clear aliasing effect. In this frequency range, uncertainties (error bars) clearly increase, probably because of the excitation of higher Rayleigh modes.

On the other hand, from 4Hz (0,25 seconds) up to 8,3 Hz (0.12 seconds) all arrays give almost the same results. Over 10 Hz, only the small size D array gives a correct non aliased velocity. For the next processing step, we just kept the mean dispersion curve calculated on the array D from 4 to 16,7 Hz. Standard deviations are also averaged in the same way.

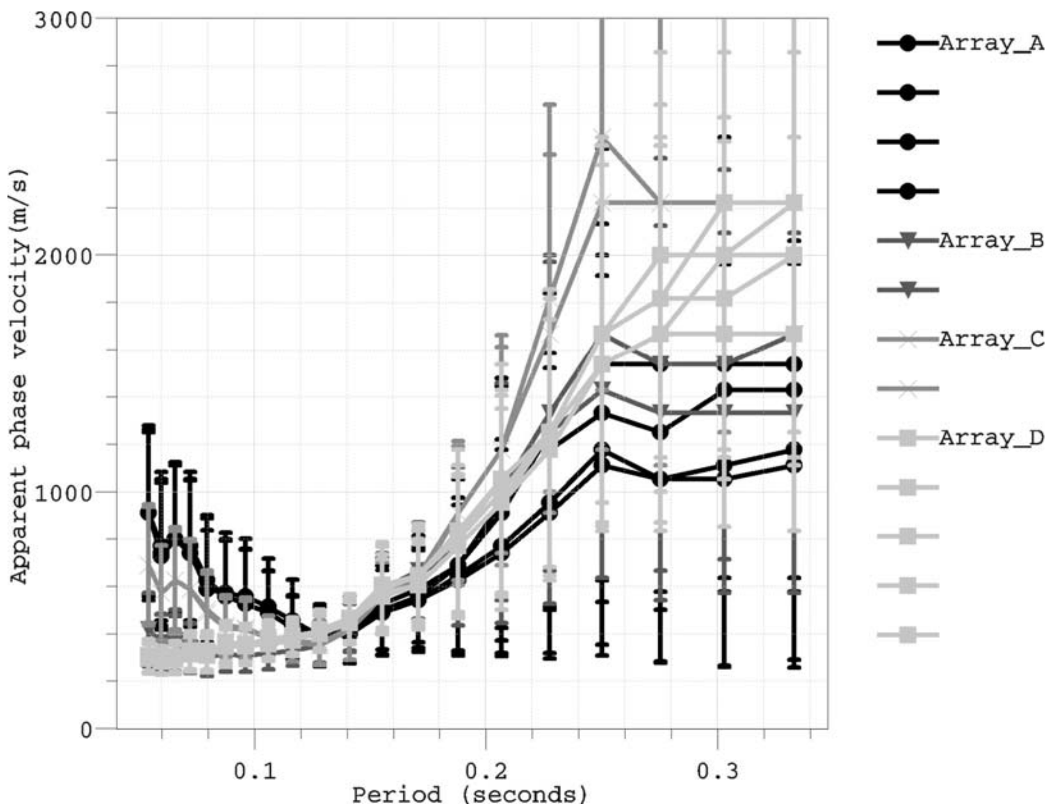


Figure 14: Dispersion curve obtained from noise arrays processed by Beam Forming FK method.

Velocity profile

Various types of parameterisations were tested: 2 layers with or without velocity gradient in the sediment layer and 3 layers. We present only the best results achieved for 2 layers with a gradient (Figure 15). On Figure 15 were plotted all the models with a misfit below 1. The misfit is the sum, for each period sample, of the squared difference divided by the standard deviation of this sample and divided by the number of samples. This means that, in general, all models with a misfit below 1 have their dispersion curve inside a one standard deviation range around the average curve to fit. Figure 15 presents the models generated for 5 distinct inversion processes initiated with different random seeds. Misfit values are lo
w with a minimum of 0,08 for the best model.

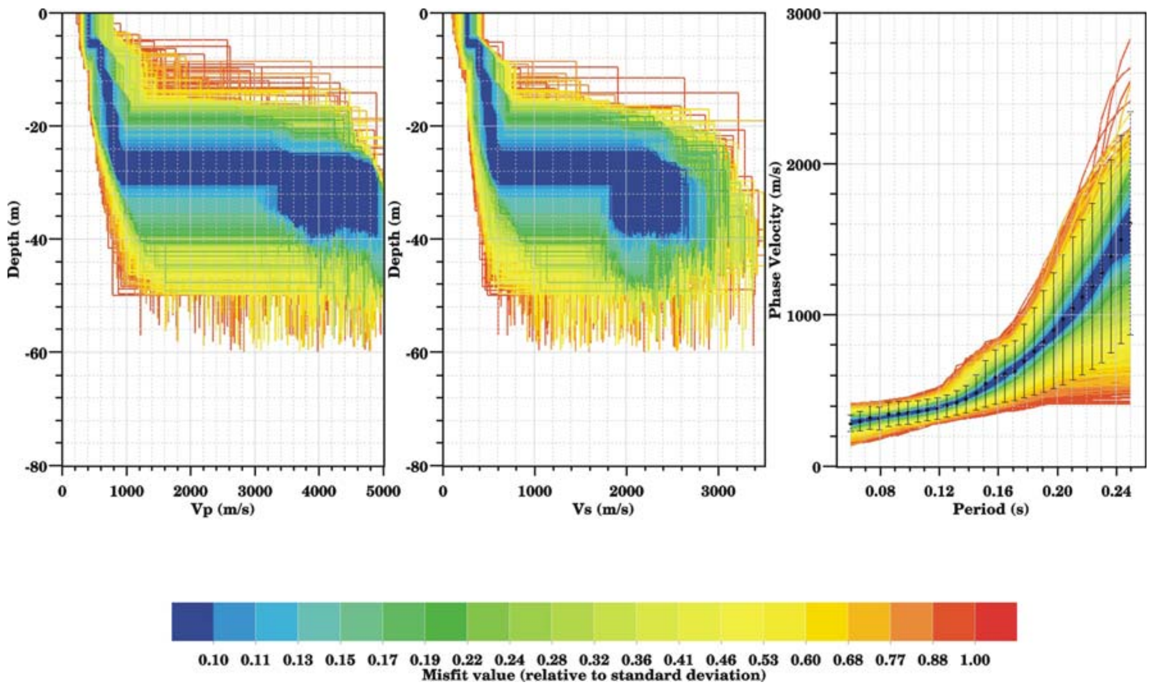


Figure 15: Inversion results from dispersion curve measured in Bavière, Liege. Case of two layers with gradient over a half space.

The depth of the top of the basement is not very well constrained and might be somewhere between 15 and 40 meters. Close to the surface the shear wave velocity is 280 m/s and increases with a power law (exponent 0,2 to 0,3) up to 550 m/s at 20 m. The uncertainties show an asymmetric distribution and can be about 100 m/s for higher values and only 50 m/s for lower values. For the same depth interval, the compressional velocity increases from 400 m/s up to 800 m/s. The Vp uncertainties are slightly greater than for Vs especially for higher values of Vp when Poison's ratio approaches 0,5.
Compared to refraction data, these results only differ for the bedrock depth assessment, which is poorly constrained by noise measurement. This point is one of the main drawbacks when using the dispersion curve alone. Several solutions are possible like the introduction of a priori information from geological data or the simultaneous inversion of the ellipticity curve.

<u>Site 2: Brussels</u>
The site is in the park of the Royal Observatory of Belgium (ROB) in Uccle (Brussels).

<u>Available data</u>
In November 2000, the ROB performed noise recordings using three concentric arrays of 4 stations with 3 components: radius_25, radius_50 and radius_100 m.
In March 2002 three more arrays were deployed on the location with a maximum radius of 130 m: ring_130, ring_25-75-130 and ring_geoph. The first two had 10 stations (Lennartz 5 seconds) each. The last one had 22 geophones (4,5 Hz of resonance frequency) distributed on a perfect circle. Figure 16 presents the map of those last three arrays.

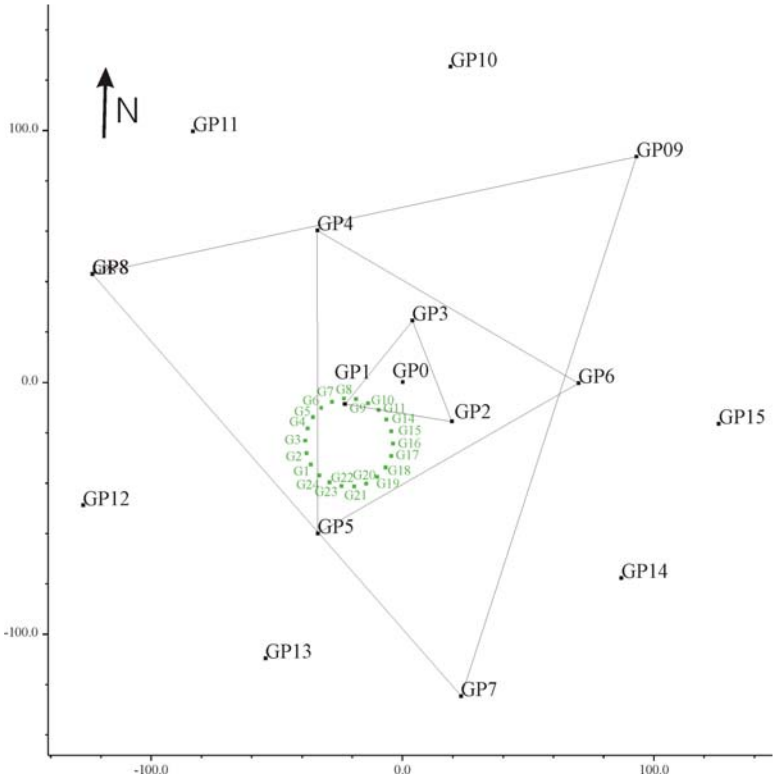


Figure 16: Site of Uccle in Brussels, array data

The array ring_130 had one central station surrounded by 9 sensors almost regularly spaced on a circle of 130 m of radius. The geometry is not perfect and the coordinates were precisely measured with a theodolite.

The array ring_25-75-130 had also one central station surrounded by 9 sensors located at each angle of three triangles, imbricated and rotated by 120 degrees, at a distance of 25, 75 and 130 m from the centre.

The radius of the array ring_geoph is exactly 18 m. This is the maximum aperture we can achieve with seismic cables 5 m between connectors (chord of the circle).

Dispersion curve

The Beam-Forming FK analysis has been applied to all 6 arrays. For the first three, 3 to 4 distinct time windows of 5 to 10 minutes have been analysed. For the last three, at least two distinct time windows of 15 minutes (ring_130 and ring_25-75-130) and of 4 minutes (ring_geoph) have been treated. All the resulting dispersion curves are displayed on Figure 17.

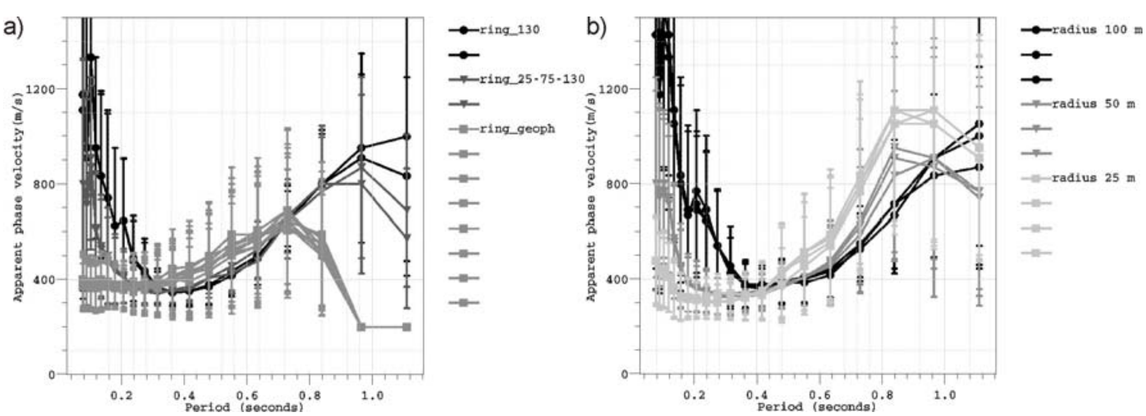


Figure 17: (a) Dispersion curves from arrays recorded in 2002, (b) in 2000

In general the stationarity with time inside each array is very good. As for the preceding site, there is a threshold frequency (resonance frequency of 1,11 Hz (0,9 s)) above which all

curves are merged. Whereas the results of ring_130 and ring_25-75-130 are almost the same on a broad frequency range, we observe a systematic decrease of the velocities calculated on the ring_geoph (figure 18a). This is probably due to the great difference between the wavelength and the array aperture (about 10 times smaller). The greatest shift is visible at a frequency far below the resonance frequency (4,5 Hz) of the sensors.
At high frequency the best curve (among the arrays measured in 2002) is the one calculated with the ring_geoph. However, a slight increase of the velocity above 4 Hz is still visible. Comparison with active source experiment showed that this is probably due to higher modes contribution rather than pure aliasing as it is the case for other radii.
For inversion, we kept an averaged curve constructed on the curves displayed on Figure 17 taking into account the valid frequency ranges for each one. This curve was defined between 1,04 Hz (1 s) and 3,1 Hz (3,3 s) with 20 samples equally spaced in period (Figure 18).

Velocity profile
The dispersion curve was inverted using a simple two layers model without velocity gradient. The figure 18 shows all generated models having a misfit lower than 1. The best model has a misfit of 0,06.
The results show that the shear velocity for the first 80 meters would be close to 370 m/s with an uncertainty of 50 m/s. The compressional velocity is greater than 500 m/s for the same depth interval. The dispersion curve does not constrain Vp when the Poisson's ratio tends to 0,5; nevertheless, we are still able to deduce a minimum value for this parameter in accordance with the results of the active sources experiments.
Vs for the deeper layers would range between 1000 and 2000 m/s. The uncertainties on Vp are too important to get a reliable estimate.
The depth of the top of the basement should be located between 80 and 140 m, a value that corresponds to observations of the borehole located inside the array limits (115 m).

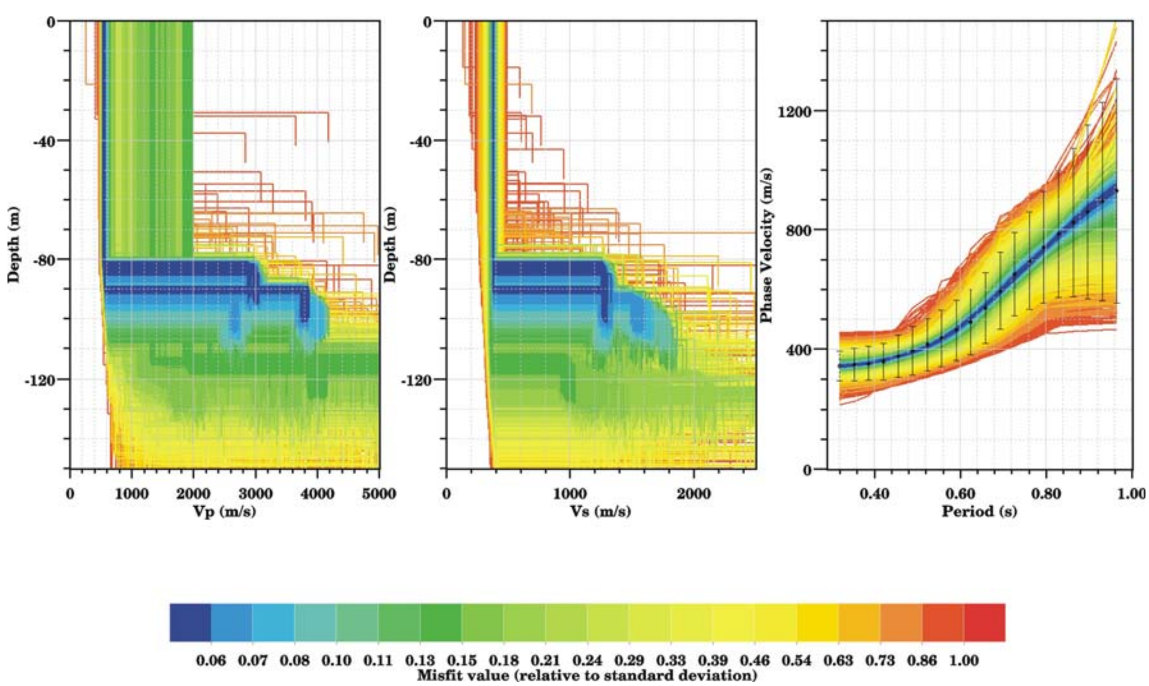


Figure 18: Inversion results from dispersion curve measured in Uccle, Brussels. Case of two layers over a half space.

Site 3: Gent

The site is inside the University of Gent.

Available data

The recordings have been made by the Royal Observatory of Belgium in 2000. We count two arrays of 4 stations, one of which is at the centre and the other three at the corners of a triangle, at 25 m and 50 m from the central station (radius_25 and radius_50).

Dispersion curve

The Beam Forming FK method applied to these arrays on 10 minutes time windows gives relatively stable results especially for the larger array. However, the coincidence of the two arrays is not perfect inside their range of validity. A difference of 50 to 100 m/s is noticed between 0,5 to 1 second. The smaller array is probably too small to correctly measure the phase delays for wavelengths around 400 m (1 Hz at 400 m/s). This 25 m array has also higher uncertainties compared to the 50 m array for the same frequency sample.

For the inversion we kept the averaged curves measured on the 50 m array between 0,4 and 1,4 seconds with 30 samples equally spaced in period.

Velocity profile

The dispersion was inverted with a simple two homogeneous layer model. Figure 19 gathers all models having a misfit below 1. The best misfit is 0,08.

Vs for the 160 first meters is about 320 m/s with an uncertainty of 60 m/s. Vp is greater than 500 m/s and probably close to 1200 m/s.

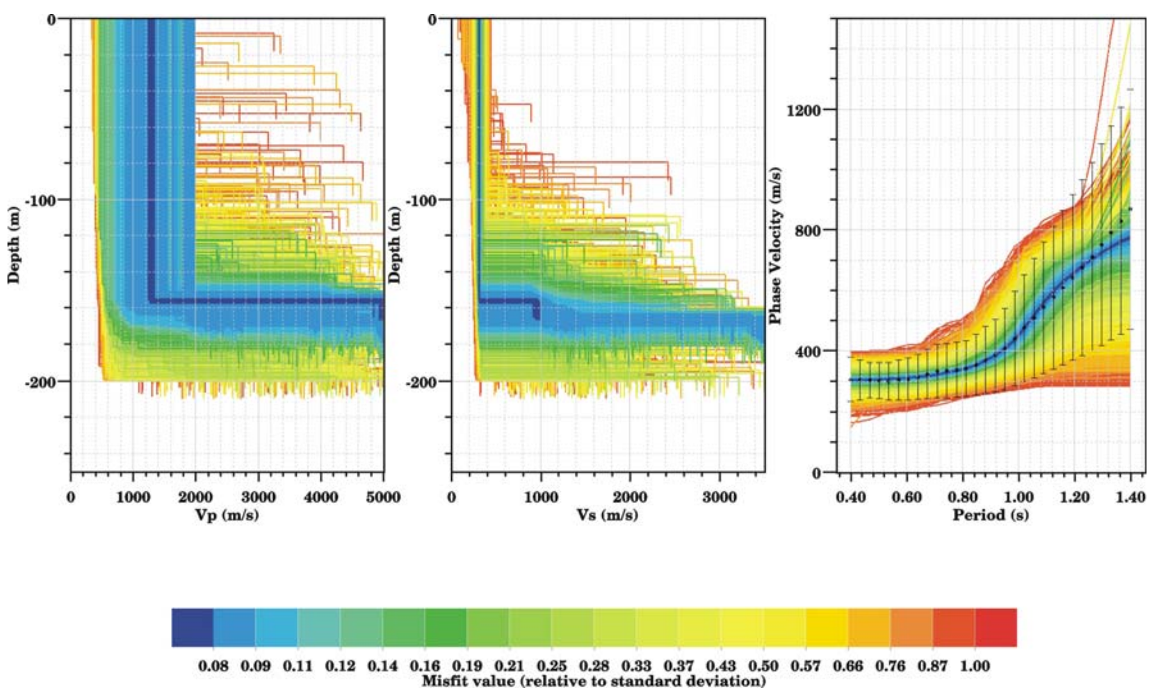


Figure 19: Inversion results from dispersion curve measured in Gent. Case of two layers over a half space.

The depth of the interface between the soft sediments and the bedrock is found around 160 m (more or less 40 m). This depth corresponds roughly to the top of the Palaeozoic basement obtained by borehole data.

Site 4: Mons

The studied site belongs to the Faculté Polytechnique de Mons.

Available data

As in Gent, two arrays of 4 stations with a similar layout were deployed by the Royal Observatory of Belgium in 2000.

Dispersion curve

The Beam Forming FK method applied to these arrays on 10 minutes time windows gives relatively stable results. The dispersion curves of the two arrays are not well defined over the whole frequency range and there is a good match only for a narrow frequency band (2 to 2,5 Hz).

For the inversion we kept the averaged curve calculated with the 50 m array between 0,48 and 0,8 seconds, and we added it to the results of the 25 m array, available between 0,22 and 0,48 seconds.

Velocity profile

The dispersion was inverted using a simple two layers model with a velocity gradient inside the sediments. The figure 20 displays all the models having a misfit below 1. The best misfit is 0,08. The measured dispersion curve is relatively well reproduced by the inverted models except at low frequencies.

The seismic shear velocity for the 60 first meters is close to 290 m/s with an uncertainty of 70 m/s (towards lower values) and 140 m/s (towards higher values). Vp is greater than 300 m/s and probably close to 500 m/s. However the Vp values obtained from refraction is 1500 m/s at a depth of no more than 10 or 15 m. Another inversion with three layers, considering the real Vp profile, gave somehow the same results for Vs.

Vs at greater depth is probably close to 1000 m/s with an uncertainty of 300 m/s.

The depth of the top of the chalks, known from geological data, is around 70 m in accordance with the average depth deduce from inversions.

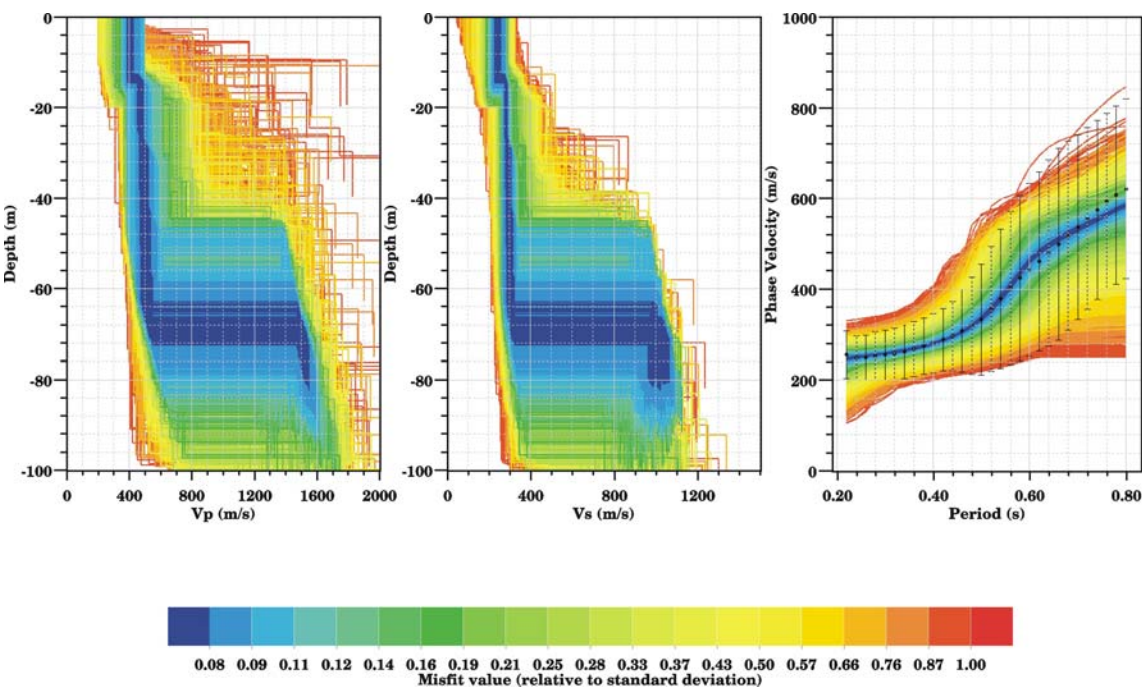


Figure 20: Inversion results from dispersion curve measured in Mons. Case of two layers over a half space.

Conclusions on the methodology

The interpretation of seismic noise arrays clearly brings reliable information (Vs values) at depth in comparison with the classical active sources methods. As all other geophysical methods, it must be calibrated and confronted with other data, especially for superficial layers, when the latter ones are available. With the statistical approach used for the inversion it becomes possible to quantitatively assess the uncertainties over all the parameters.

The association of all the existing and new data allows us to define the 1D structure for the 4 investigated sites, in terms of Vs variation with depth. This information has been used for computing the theoretical 1D transfer functions.

3.2.3. Site effects in Brussels, Gent, Liege and Mons

The transfer function of the site effects for the four studied sites are presented on figure 21A. For each site, three curves are indicated. The first one corresponds to the H/V ratio. The two other one's are the transfer function calculated using respectively the average soil characteristics with depth (1-D model) and a Monte-Carlo modelling considering the uncertainties in the soil parameters.

For two sites, Gent and Brussels, the 1-D modelling provides a theoretical transfer function well correlated with the first peak in the measured H/V ratios. In the two other sites, Liege and Mons, the measured H/V amplitudes are higher than the calculated one's. It is clear that in all the cases, the H/V method furnish a very stable frequency peak, which is well correlated with the resonance frequency of the studied sites.

Considering the 1-D modelling, the site in Liege is characterized by a broad peak centred at a frequency of 4.2 Hz corresponding to a maximal amplification of 4.6. The site in Gent has a predominant narrow peak at 0.52 Hz with an amplification close to 7, whereas the site in Brussels has a peak at 1 Hz with a maximal amplification of 6.6. The site effect in Mons is more complex. It is characterized by different peaks with maximal amplifications from 4 to 5 at frequency ranging from 0.52 to 6 Hz.

For each site, accelerograms on the bedrock corresponding to the two reference earthquakes have been convolved with the site effect time function to obtain the resulting strong ground motions at the soil surface. An example of these computations is presented on figure 21B. It corresponds to the site in Gent for a magnitude 5.5 reference earthquake having occurred at 50 km far from the site.

3.2.4. Soil elastic response spectra

Based on the available geological and geophysical information, each site has been classified in term of the EUROCODE-8 soil classes. The sites in Gent, Brussels and Mons are characterized by a succession of some dozen of meters of sandy-clay deposits with S-waves velocity ranging from 180m/s to 360 m/s. Thus, they belong to the class C of soil. The site in Liege, characterized by at least 5 m of alluvial deposits with V_S lower than 360 m/s, has been classified as soil E, even if it could be also enter in class C.

Figure 22 to 25 show the elastic response spectra for each site for the two reference earthquakes. The EUROCODE-8 spectra corresponding to the corresponding soil category are also indicated.

For the site in Liege, the spectra are very close to that of the EUROCODE-8, except at the frequency of the site effect where the amplitude is four times higher than the value of the EUROCODE-8. For the Brussels and Gent sites, the spectra show similar behaviour. They are less than the EUROCODE-8 values at frequencies greater than 10 Hz. Between 2 Hz and 10 Hz, the values are higher than the EUROCODE-8. For lower frequencies, and for the two reference earthquakes, they are higher than type 2 norm but coincide with type 1 norm. Thus, for these two sites and for the Brabant Massif as a whole, it appears important to take into account the site effects. A rough solution could be to consider the envelope of the two EUROCODE-8 spectra as the elastic response spectra in the National Document of Application for EUROCODE-8.

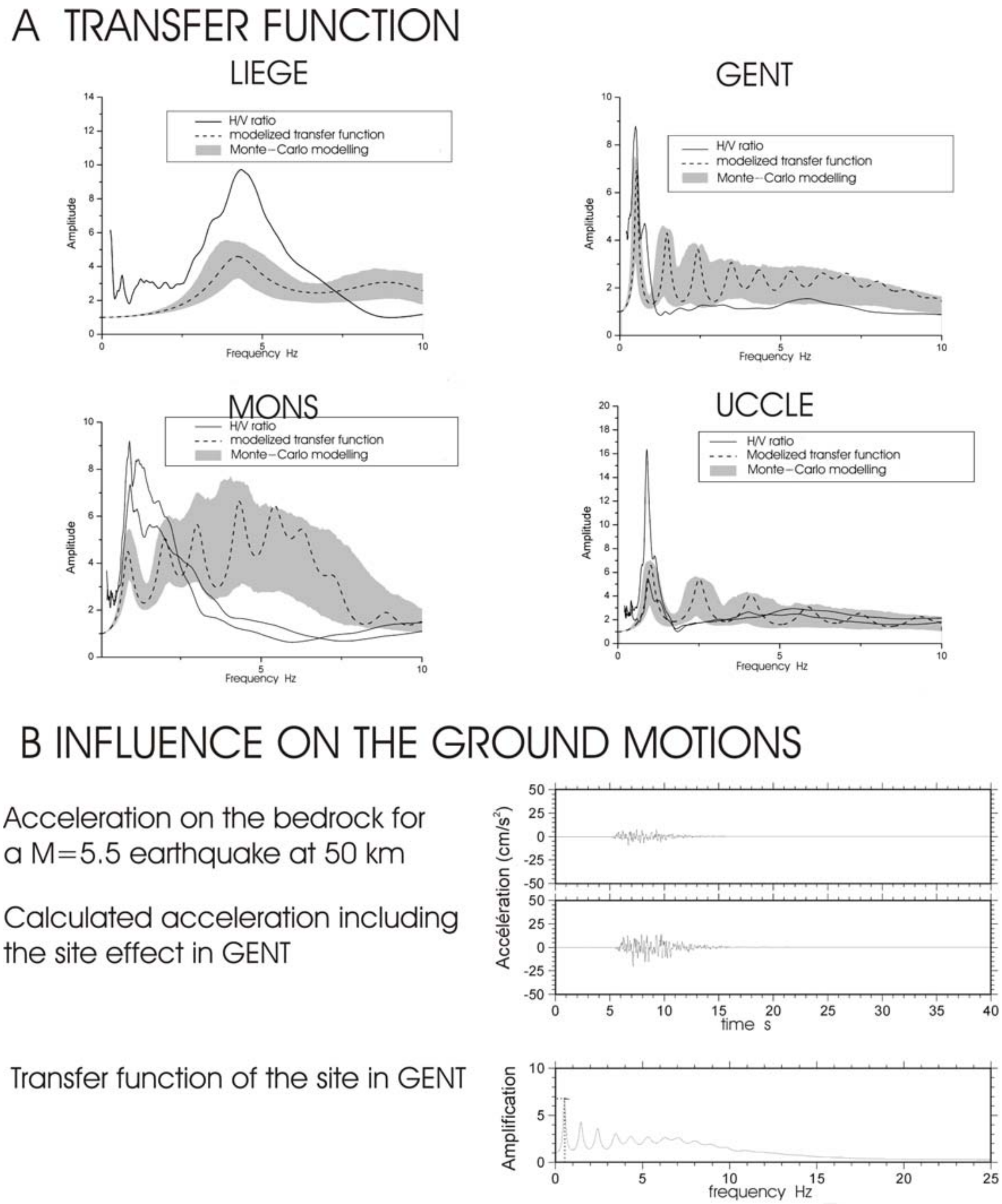


Figure 21: A. Transfer function for the four studied sites - B. calculated acceleration at the ground surface for the Gent site resulting from an M=5.5 reference earthquake**.**

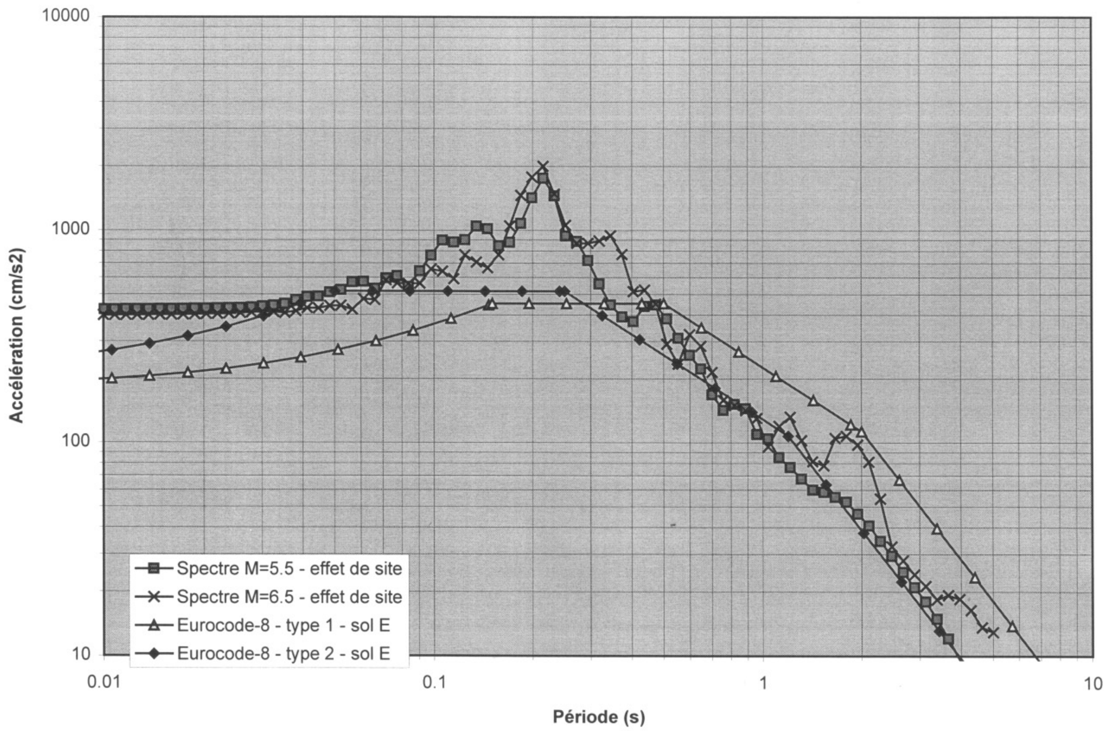


Figure 22: Elastic response spectra at the surface for the Liège site.

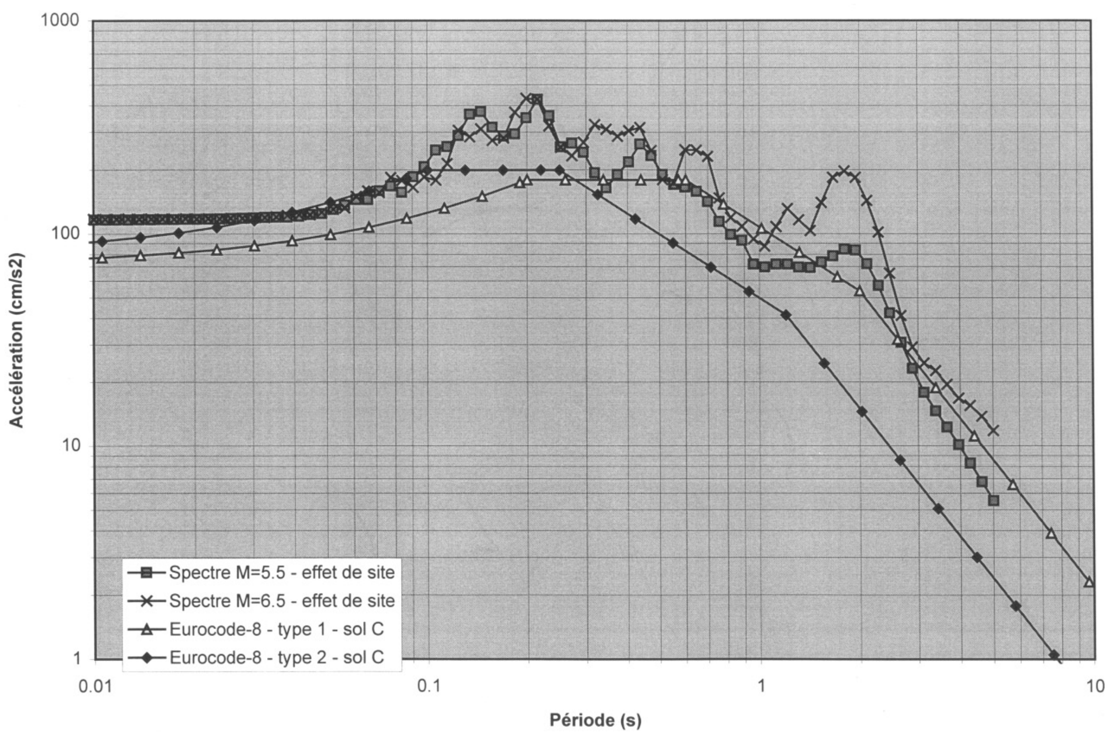


Figure 23: Elastic response spectra at the surface for the Gent site.

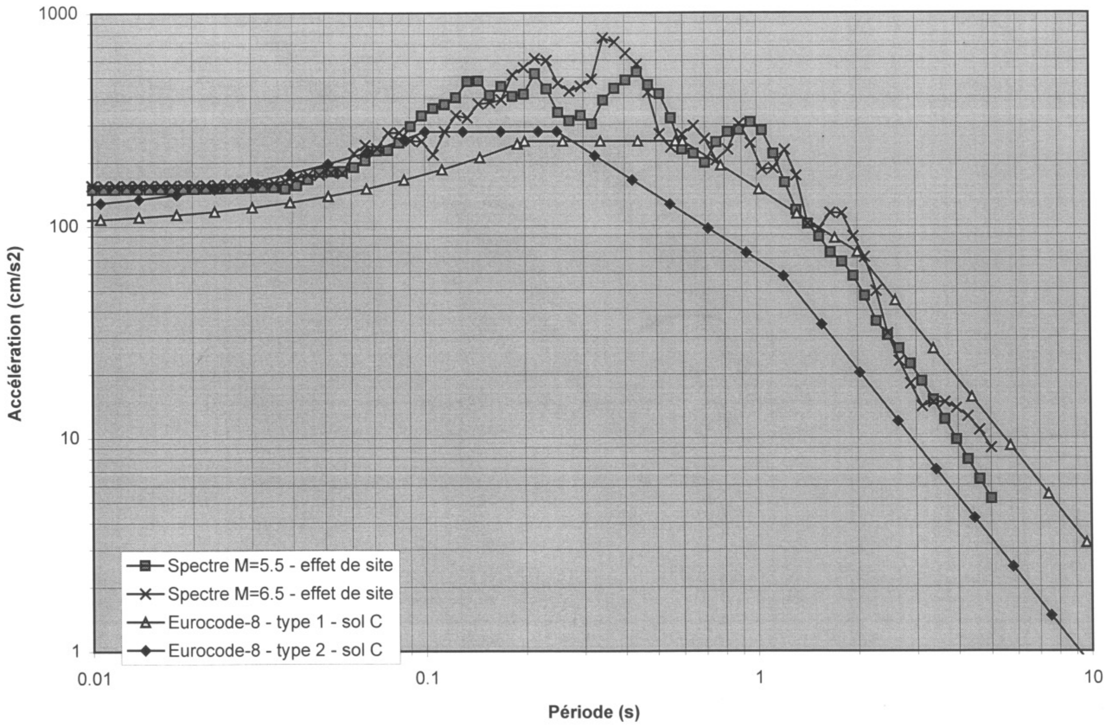


Figure 24: Elastic response spectra at the surface for the Brussels site.

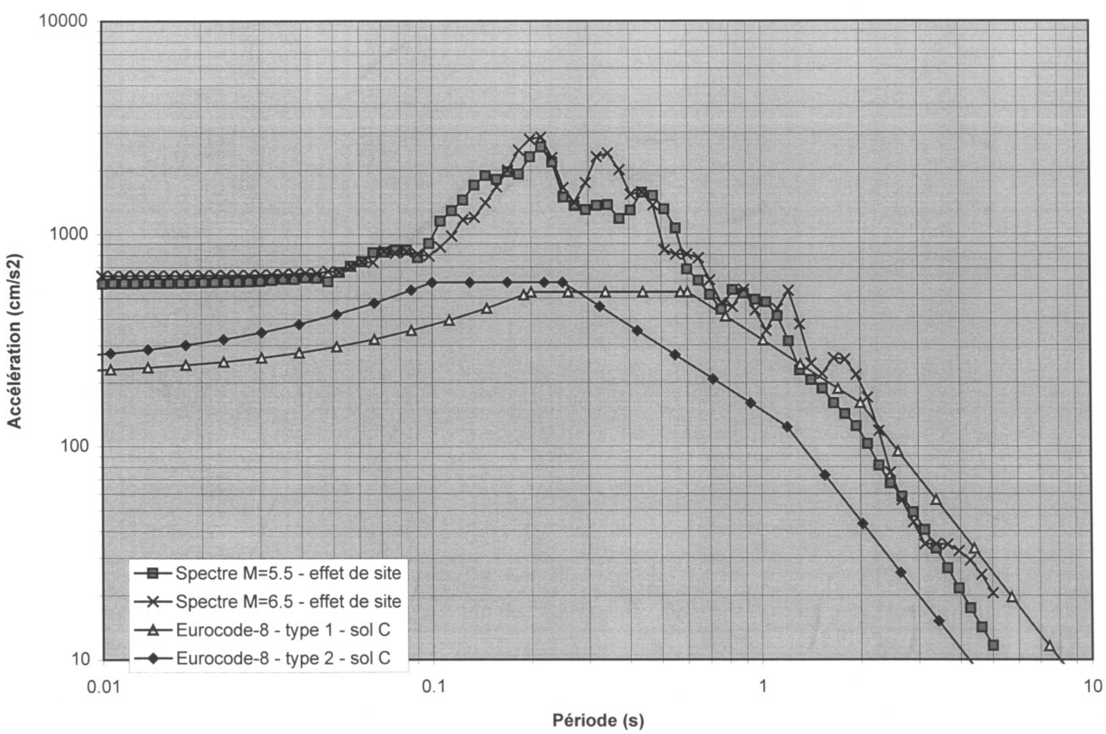


Figure 25: Elastic response spectra at the surface for the Mons site.

For the site in Mons, the calculated response spectra have very high values, mainly for frequencies greater than 3 Hz. The region of the Mons basin should be studied more intensively in the future, because the study suggests that site effects could have there disastrous effects on the strong ground motions during future strong earthquakes.

As a conclusion of these modelling, it is now evident that regional site effects in the north of the country should be taken into account by a convenient choice of the response spectra in the National Document of Application for EUROCODE-8.

3.3. Vulnerability in low seismicity zones

Two types of results are obtained from the vulnerability study of non-engineered structures in low seismic activity zones as Belgium:

- Values of the connection forces between constructional elements (walls, floors, roofs). The values to be considered in the design of the links between the constitutive elements are explained and accompanied by recommendations.
- Drawing of connecting details between constitutive parts and their layout in the various critical parts of the buildings.

The results are presented and explained in a technical guide (Plumier et al., 2003) written in French. The main aim of this technical guide is to give wide information on the problem of seismic design in Belgium to the architects, to the construction undertakers, to the design offices and engineers.
The final aim is to convert this technical guide and introduce all the collected knowledge about seismic behaviour of structures in low seismicity regions in a bi-lingual technical handbook edited by the BBRI (Belgian Building Research Institute).

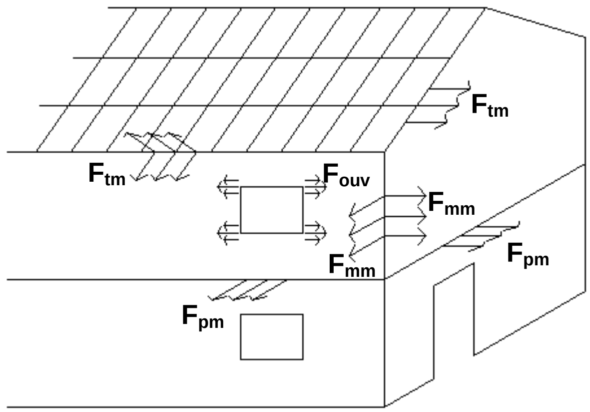
As secondary results, we can mention the education of a young engineer in the domain of masonry structures and the development of a new competence in the seismic research team at University of Liege.
Another interesting feature is that the results are at the base of the definition of test specimens (2 small houses) to be tested in 2003 on shaking table at the laboratory of LNEC Lisbon (Portugal) under funding of the large installation ECOLEADER European project. One house will be built following the Belgian practice without any specific design. The other will incorporate specific design recommended by the present study.

The interest of the obtained results is that they provide an estimate of the forces to take into account for the seismic design of usual masonry structures like dwellings. Coming from a situation in which no design value existed, these results are a serious step ahead and will for sure be very useful. The results are presented in a technical guide, which does not only give drawings, but also summarises in a simple way the basic seismic design principles to allow the reader to understand and use the results in a wise manner. This way of doing was based on the comparison with other existing guides as French guides AFPS (2001), AFPS (2002), an Australian Guide and the Utah Guide (Epicenter, 1992).

The main results are summarised hereafter.

3.3.1. Forces in the wall-to-wall links

Figure 26 gives an overview of the set of links considered.



mm : wall-to-wall,
pm : floor-to-wall,
tm : roof-to-wall,
ouv : openings

Figure 26. Representation of the different link forces

The values of the forces in the wall-to-wall links F_{mm} (cf. Figure 26) are function of the degree of link of the floors to the walls. So, we distinguish at Table II three types of floors and the corresponding wall-to-wall forces.

Table II. Forces F_{mm} at the wall-to-wall links

Type of floor	F_{mm} average [kN/m]	F_{mm} peak [kN/m]
Diaphragm floors linked on the four sides	15	30
Diaphragm floors linked on two sides	20	80
Floors with main timber beams in one direction, without diaphragm	30	120

To design the link, it is recommended

- to consider the peak value at the top of the wall when no floor is provided, on a minimum height of two mortar layers
- to consider the average value everywhere else

It is interesting to see that the less the floor is linked to the walls, the higher the wall-to-wall link forces are and the higher the concentrations (peak) of forces are. A low degree of connection of the floors to the walls allows the movement of the wall in the direction perpendicular to their plan and causes high connection forces between walls.

It is recommended to always realise the connection of the floors on the four sides, to reduce the values of F_{mm} and ensure a real behaviour of the structure as a closed box.

3.3.2. Forces in the floor-to-wall links

The values of the forces in the floor-to-wall links F_{pm} (cf. Figure 26) depend naturally on the degree of connection of the floors to the walls. Table III gives the floor-to-wall forces for the three types of floors.

Table III. Forces F_{pm} at the floor-to-wall links

Type of floor	F_{pm} average [kN/m]	F_{pm} peak [kN/m]
Diaphragm floors linked on the four sides	10	30
Diaphragm floors linked on two sides	20	110
Floors with main timber beams in one direction, without diaphragm		25

In the case of diaphragm floors, the maximum values of forces correspond generally to the corners of the openings (stairs). To design the links, it is recommended

- to consider the peak value near the discontinuities
- to consider the average value everywhere else
- to always realise the connection of the floors on the four sides in order to reduce the values of F_{pm}.

In the case of diaphragm floors linked on the four sides, one unique value of the force may be considered, with or without discontinuities. This value is taken equal to 20 kN/m. This simplification is proposed to be in accordance with the value proposed in Eurocode 2 - prEN1992-1 (2001) § 9.9- for the tying systems (horizontal ties to walls), which gives the single value of 20 kN/m.

3.3.3. Forces in the roof-to-wall links

For what concerns the design of the link between roof beam and wall, the results are not as clear as for the other links.

We know that the forces are directly function of the stiffness in plan of the roof, but at present, it is impossible to obtain a reliable assessment of the stiffness of a roof by taking into account all its components: main elements (beams, purlins, rafters), « under-roofing », roofing,… For the optimal behaviour, the stiffening of the roof to obtain a good diaphragm has to go hand in hand with a distribution as uniform as possible of the bearing reactions under seismic action on the walls that support this roof. Two ways are possible to stiffen the roof in its plan:

- by adding bracings (diagonal or V-bracings). This way is very effective, but may lead to high concentration of forces on the gable wall, where the bracings are anchored. It can then be difficult to transmit these important local forces without damage to the masonry.
- by fixing efficiently a rigid « under-roofing » (nailed panels for example) to the structure of the roof (purlins, rafters).

The second parameter that influences the link forces between roof beams and gable walls is the slenderness of the gable wall itself. The greater the slenderness of the gable wall is, the more important the forces between the roof beams and the gable wall are.
The slenderness of the gable wall depends on the position of the floor under the roof that plays the role of diaphragm. The free height of the wall is smaller if the floor is near the roof. (see Figure 27), and consequently, the mass of the wall that induces forces in the links between roof and gable wall is smaller. The present study has not succeeded in assessing the favourable effect of the decrease of the slenderness of the gable wall. For security reasons, the values of the forces given below don't consider this favourable effect.

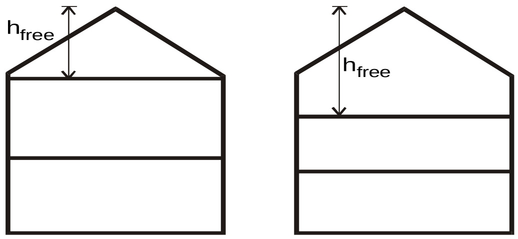
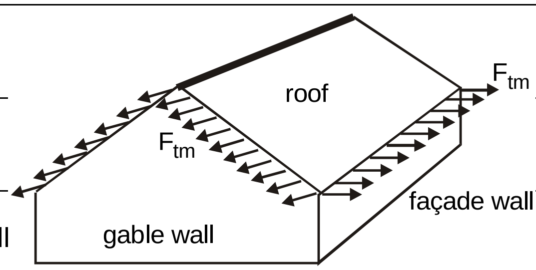


Figure 27. Influence of the position of the floors on the slenderness of the gable walls.

All these considerations, the numerous possible situations and the lack of knowledge for what concerns the real stiffness of the roofs lead to recommend for the design of the roof-to-wall links for all types of structural roof systems the values of the forces given at Table IV.

Table IV. Forces F_{tm} at the roof-to-wall links.



Link	F_{tm} average [kN/m]
Roof – to - gable wall	10
Roof – to - façade wall	15

3.3.4. Forces in the walls near the openings

The values of stress concentrations above and below the openings (doors, windows) are estimated at 40 kN/m, at the most. The reinforcements have to be laid in minimum 2 mortar layers above the opening and in 2 mortar layers below the opening. The length of the reinforcement is taken equal to the width of the opening plus 2 times 40 cm (40 cm at each side of the opening).

3.3.5. Influence of the soil and of the seismic zone

The values of the forces given in Tables II-III-IV may be adapted in function of the acceleration at bedrock (zone 1 or zone 2) and in function of the type of soil (class A to E) by applying a decrease/increase proportional to the decrease/increase of the acceleration of the "plateau" of the design spectrum.

As the forces found in the study are orders of magnitude, simple adaptation coefficients can be proposed. They are given in the following table:

Table V. Coefficient by which the forces F_{mm}, F_{pm} and F_{tm} should be multiplied in function of the seismic zone and the soil class.

Class of soil	ZONE 2 a_g = 1 m/s²	ZONE 1 a_g = 0,5 m/s²
A	0,65	0,35
B-C-D-E	1	0,50

3.3.6. Detailing

From the forces summarised in the above tables, the required connecting devices between the elements (links of horizontal to vertical elements, links of vertical to vertical elements, etc…) may be defined and designed. The forces being related with the type of diaphragm, some solutions are provided to make diaphragms effective.

Different technological possibilities of connections have been considered and presented in a technical guide (Plumier et al., 2003). It is clear that the presented details are examples of possible solutions and that other solutions can be valid.

The existing technical guide provides the principles that have to be applied to obtain box behaviour of the structure, but also some prepared solutions, to be applicable directly without further calculations. The chapter concerning the technological details is divided in several parts:

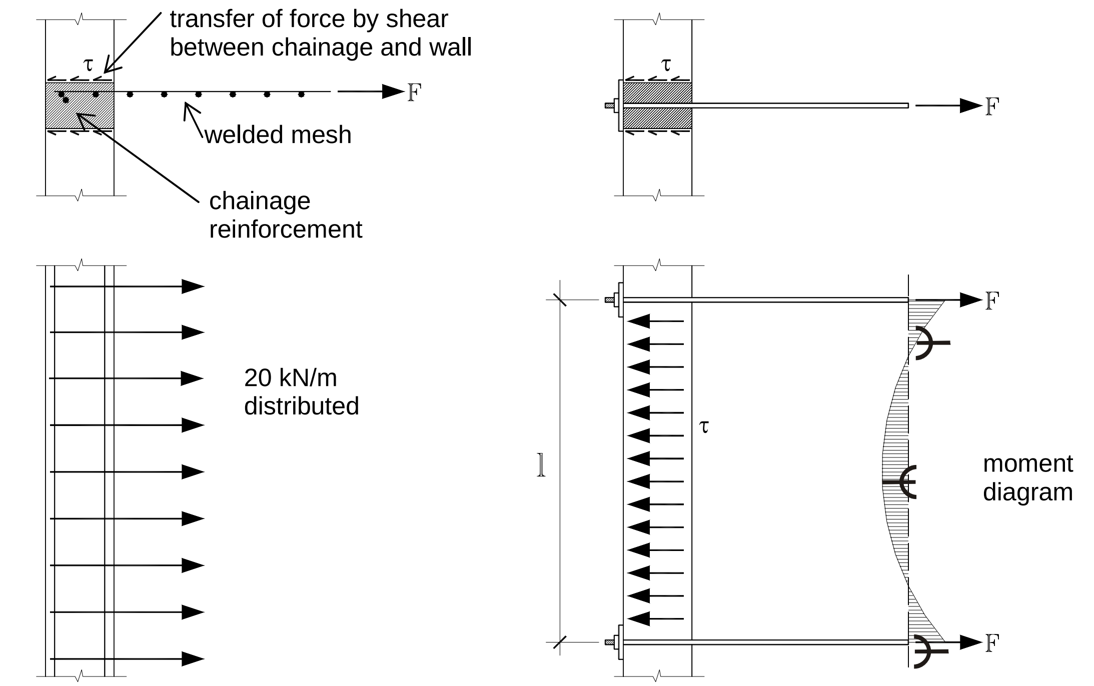
- Tying systems
  - Links between floor and wall. Principle of the transfer.
  - Horizontal tying systems.
- Diaphragms
  - Rigid diaphragms
  - Partially rigid diaphragms
  - Beams grid as diaphragm
- Links between concrete floors and walls (24 Figures)
- Links between timber floors and walls. Stiffening of timber floors (8 Figures)
- Links between roof and walls. Stiffening of roof (11 Figures)
- Links between walls (4 Figures)
- Reinforcing at the lintels and openings (2 Figures)
- Chimney and non-structural elements (2 Figures)

Some examples will be explained hereafter.

<u>Tying systems or chainages</u>
In a box structure, the forces between the walls of the box are tensions and compressions perpendicular to the junction lines. These junction lines and the connections between the different planes (walls, floors) have to be configured to ensure the transfer of these forces. *This function does not imply that the junction lines must absolutely be tie beams with classical detailing, like reinforced concrete beams with four longitudinal bars and stirrups.* The junction lines have only to be "chainings" or chainages, that means elements able to withstand tensions along their axis and able to ensure the transfer of tension/compression perpendicular to their axis.

With this definition, the need to give more or less stiffness to the chainage element will depend on the way the transfer of the forces perpendicular to the axis of the chainage is realised.

- if the transfer is well distributed on the chainage, this chainage does not need much stiffness. Example: the connection of a welded mesh to a wall, realised by hairpins placed in overlapping with each bar of the reinforcing mesh and spaced of 150 to 200 mm - Figure 28(a).
- if the force is transferred by concentrated elements located far from each other, the transfer of force to the chainage needs a stiffness of the chainage adapted to the needs of the transfer of force from the chainage to the other concerned plane. Example: floor-to-wall connection ensured by floor timber beams spaced of more than 500 mm – Figure 28(b).



Bending moment of the chain: 0
Required stiffness: 0

(a)

Bending moment of the chain:
$M^- = 20 \text{ kN} \times l^2/12$ $M^+ = 20 \text{ kN} \times l^2/24$

(b)

Figure 28. Transfer of forces perpendicular to the chainage (a) transfer of well distributed forces by not much spaced re-bars (b) transfer of forces by concentrated spaced elements

<u>Principle of the transfer of forces between floor and wall</u>
Figure 29 gives details of the different mechanisms of the possible transfer of forces between floors and walls.

At Figure 29(a), the transfer of force is only ensured by shear. It is the minimal solution.
In this case, the horizontal dimension of the chainage element is linked to the resistance of the masonry in shear. The design resistance is: $f_{vko}/\gamma_M = 0{,}2/2{,}5 = 0{,}08$ N/mm².
To transfer 20 kN/m (see Table III), the area A where the shear resistance is mobilized has to be:

$$A \geq 20\,000/0{,}08 = 250\,000 \text{ mm}^2 = 0{,}25 \text{ m}^2.$$

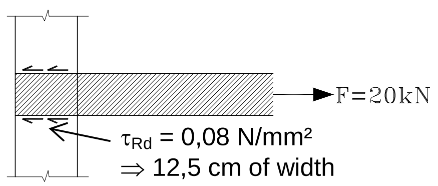
If 2 failure surfaces are mobilized, a masonry width of 12,5 cm is necessary.
If only one surface is mobilized, the required masonry thickness is in principle 25 cm.

At Figure 29(b), a vertical connector is introduced in the wall, which allows obtaining a more secure solution by transferring the force to full mortar joints presenting less risk of defect.

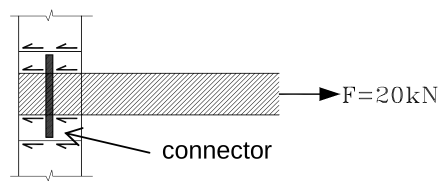
At Figure 29(c), distribution elements are introduced at the bottom of the wall. This solution is necessary to activate the chainage
a) in case of concentrated tension applied to a chainage of small stiffness: distribution element should be stiff and possess enough horizontal length.
b) in case of concentrated tension applied to a chainage distributed in height (reinforcing bars in the horizontal mortar joints): distribution element should be stiff and high.

Finally, the transfer of force may also be ensured by taking into account the friction in the walls that are highly vertically loaded – see Figure 29(d)
The friction resistance τ_R is function of the compression stress σ and of the friction coefficient μ.
If σ = 0,5 N/mm² and μ = 0,4, $\tau_R = \mu.\sigma$ = 0,2 N/mm²
If the security coefficient for masonry γ_M = 2,5 is taken into account,
τ_{Rd} = 0,2/2,5 = 0,08 N/mm², similar to the shear resistance.

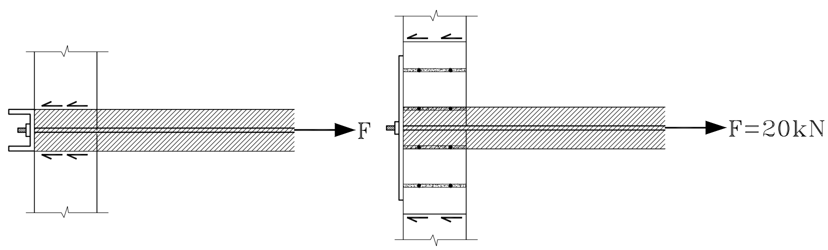
At Figure 29, each mechanism is represented. In practice, it is cautious to activate several mechanisms.



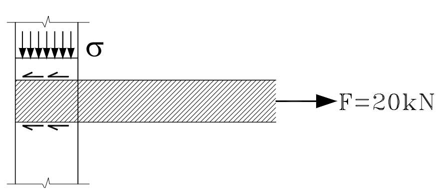
(a) Transfer of force by shear



(b) Transfer of force by shear + connector



(c) Transfer of force by shear + distribution element at the bottom



(d) Transfer of force by friction

Figure 29. Floor-to-wall link. Principle of the transfer of forces

<u>Horizontal tying systems</u>
The axial tension force in the chainage depends on the effectiveness of the horizontal diaphragm. In case of undeformable diaphragm, the transfer of the horizontal forces may be done by uniform shear from the floor to the vertical walls. Ideally, no tension occurs in the longitudinal chainage. In case of a partial diaphragm, the transverse forces are transmitted through the partial horizontal diaphragm to the longitudinal chainages and finally to the vertical walls (see Figure 30).
The axial tension force in the chainage is obtained by equilibrium of the transverse 20 kN/m forces on the chainage of the wall in the perpendicular plane.
If the length of the wall l_y = 7 m (l_y is defined at Figure 30), the application of 20 kN/m in the x direction produces an axial force of 20 x 7/2 = 70 kN in the chainage, which justifies the value proposed in Eurocode 2, valid for a length of wall smaller than 7 m.

Consequently, the required section is equal to 70000/500 = 140 mm², which can be ensured indifferently by 1 ϕ 14, 2 ϕ 10 and 3 or 4 ϕ 8 in S 500 steel with high bond.
Figure 31 gives some possible sections of horizontal chainages.

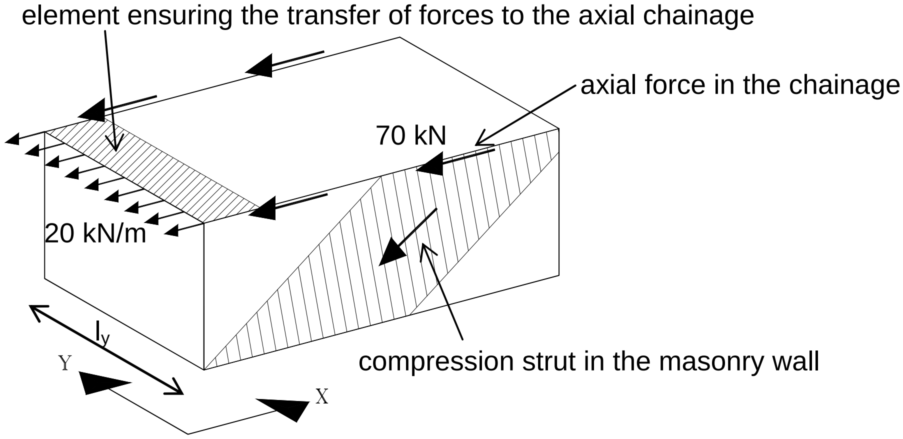


Figure 30. Necessity of axial resistance of the chainage in presence of partial diaphragm. Case of transfer by a prefabricated concrete element.

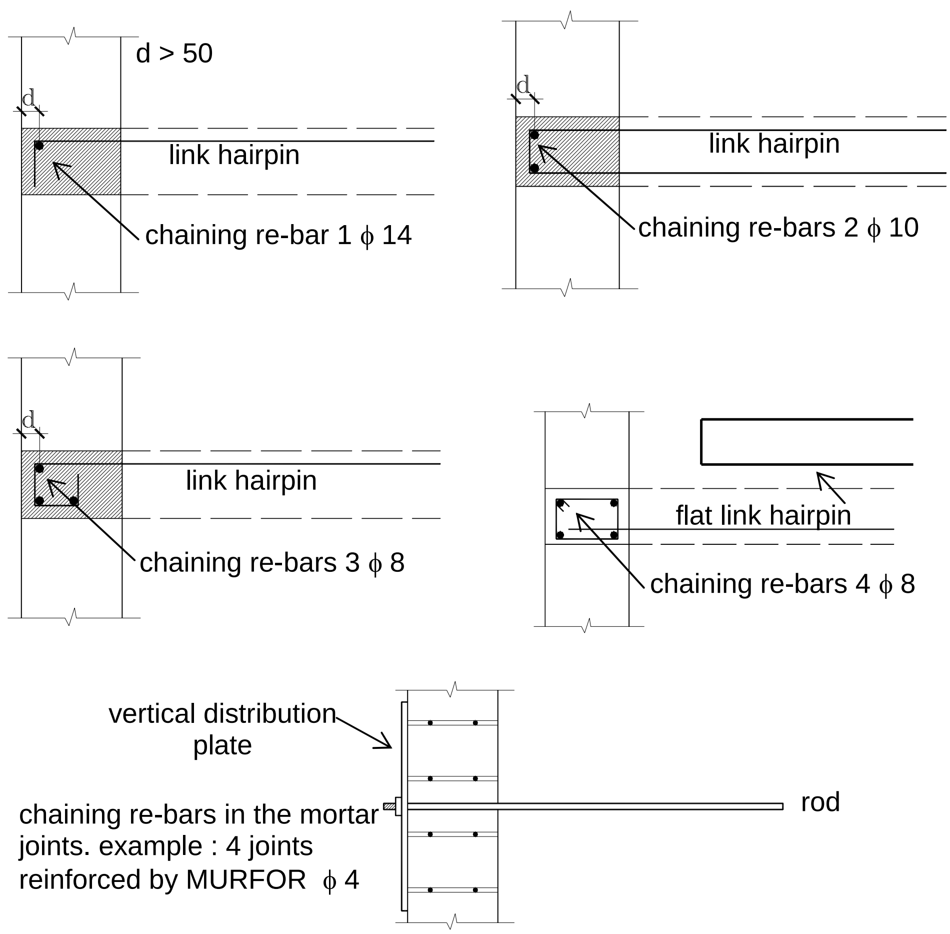


Figure 31. Possible solutions of horizontal tying systems

<u>Diaphragms</u>

The diaphragms are used:

- as horizontal support for the vertical walls; they sustain forces perpendicular to these walls (20 kN/m at the wall/floor junctions, for example), in tension and in compression.
- as transfer elements of these reactions to the vertical walls parallel to the exterior applied forces; these walls, stressed in their plan, offer a good capacity to transfer the horizontal forces to the foundation.

In the context of the non-engineered simple buildings, the function « diaphragm » can be practically realised in different ways.

Undeformable diaphragms

To behave as an undeformable block, the floor can be constituted by, for example:

- a concrete slab cast in place and reinforced in two directions;
- prefabricated concrete elements (independent) covered by a concrete coating reinforced in two directions
- prefabricated concrete elements (independent) linked by rods in the direction transverse to the elements ; in this case, for a seismic action in the x direction – Figure 32 -, each prefabricated concrete element, by its work in bending in the horizontal plan, transfer to its supports the force that reaches it. The rods ensure that all the elements work together.
- for a seismic action in the y direction, shear has to be transmitted between the independent elements, to avoid the deformed shape of the type represented at Figure 33. This can be ensured by a correct infill of the joints between the elements with a concrete of good resistance, with the additional condition that the borders of the elements possess indentations ensuring a good hanging of the cast in place concrete.
- a set of timber beams covered by nailed plywood panels - Figure 34. In this case, the beams that go from one side to the other side in the x direction may play the role of tension trusses. This is not the case of the cross pieces, which have to be doubled by steel tendons for the transfer of tension or be connected to the beams in such a way that tension can be transferred.

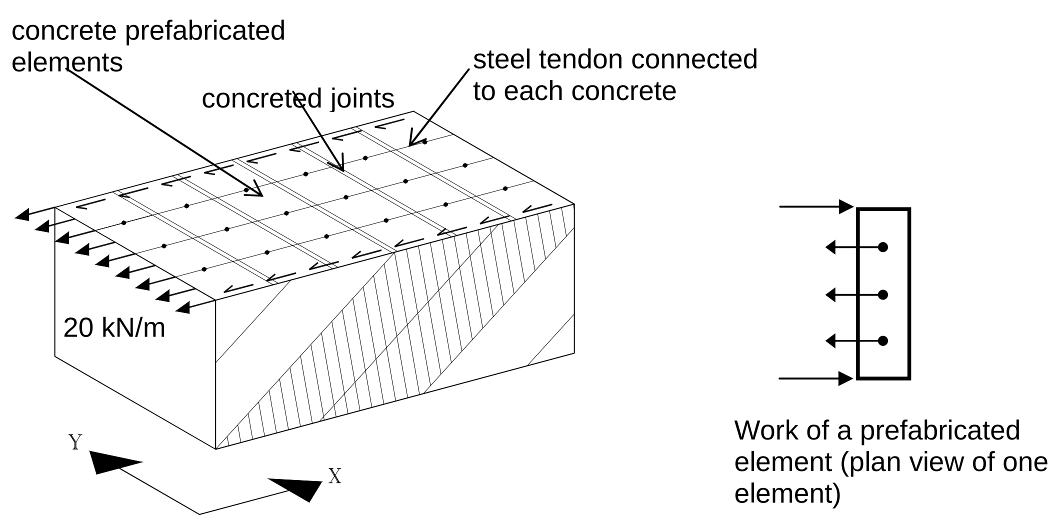
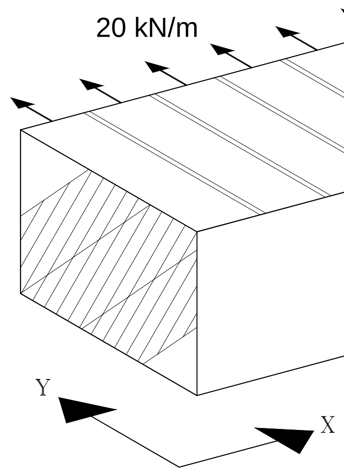


Figure 32. Floor constituted by concrete prefabricated elements. Seismic action in the x direction.



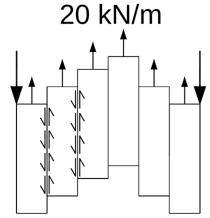


Figure 33. Floor constituted by concrete prefabricated elements. Seismic action in the y direction.

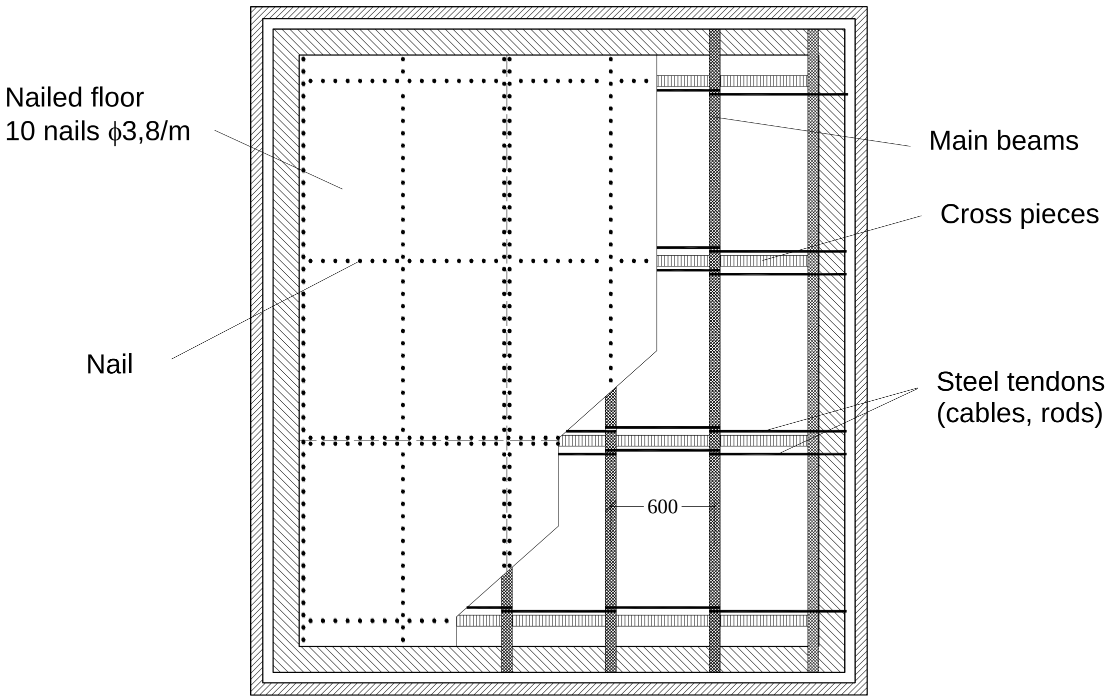


Figure 34. Timber floor constituted by beams (x direction), cross pieces and steel tendons (y direction) and covered by plywood elements nailed to the beams and cross pieces

Partially undeformable diaphragms

It is not always necessary to realise a complete floor working as an undeformable block. The stiffness of smaller parts of the floor may be sufficient to transfer the horizontal forces to the vertical walls parallel to the applied action. In this case, an axial force is transmitted in the longitudinal chainage (see Figure 30), which may be not negligible. It is the case for

- a floor made of prefabricated concrete elements, without concrete coating with mesh or without trusses perpendicular to the prefabricated elements (Figure 30).

- a timber floor without nailed panels, but with timber bracings in each direction (see Figure 35)

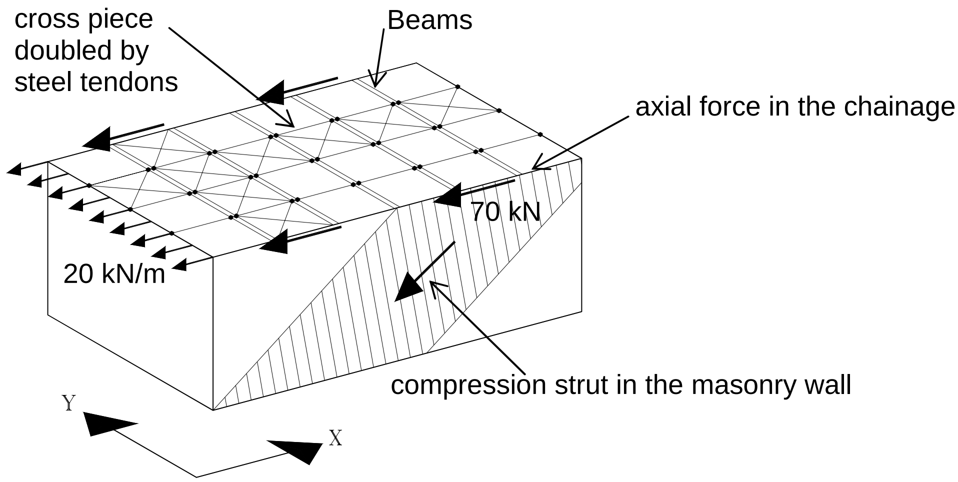


Figure 35. Partial diaphragm. Case of a timber floor with beams and cross pieces (without nailed panels), stiffened by truss braces made of timber elements and steel tendons.

Grid of beams as diaphragm

The transverse stiffness of the beams of a floor positioned in the y direction may be used to transfer the horizontal forces from the x direction to the vertical walls parallel to the applied action of x direction. The beams in the y direction should be associated by elements transferring tension (tendons, see Figure 36).

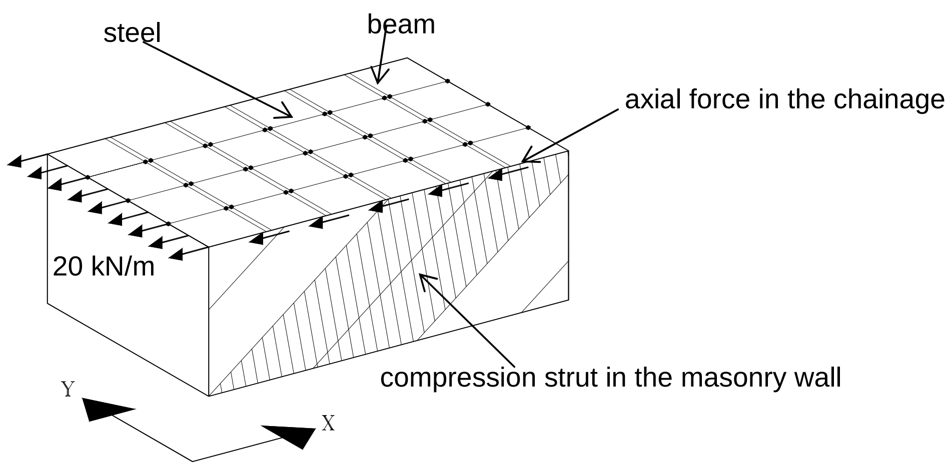


Figure 36. Grid of beams as diaphragm

In the y direction however, this arrangement does not ensure any diaphragm effect (see Figure 37), unless the tendons are doubled by cross pieces supporting compression and unless a lattice beam is constituted in the x direction (see Figure 35). This lattice beam is necessary because the cross pieces do not offer individually a transverse stiffness able to resist to the forces in the y direction.

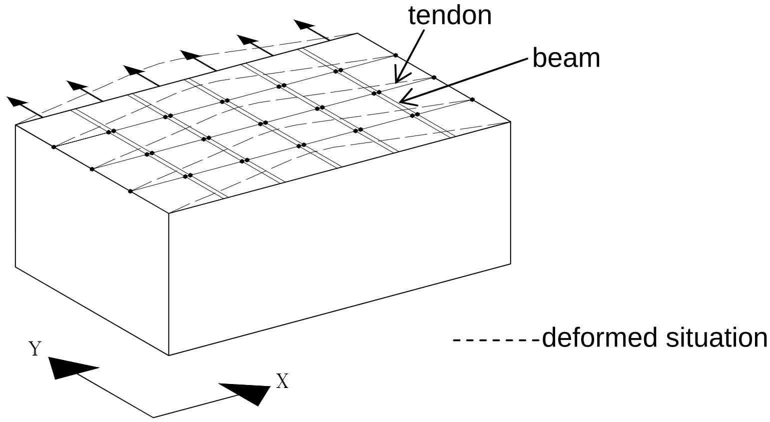


Figure 37. No diaphragm effect in the absence of nailed panels

Links between concrete floors and walls

Some examples of specific drawings proposed in the technical guide are given at the Figures 38 to 43 for the links between concrete floors and walls. The described solutions are given for links between floors and peripheral walls, but have to be applied for the internal walls of the building when the floor is not continuous. The drawings are given for one type of prefabricated concrete elements. They are generally adaptable to all prefabricated concrete support.

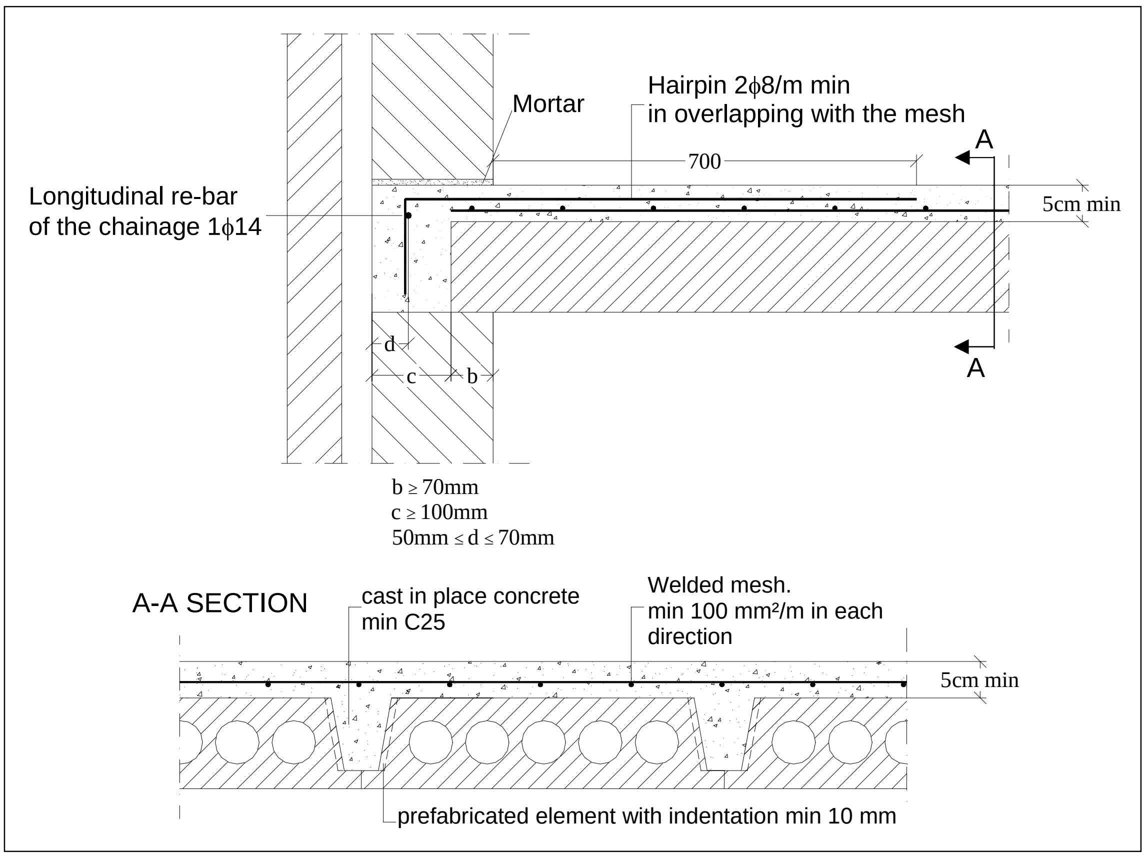


Figure 38. Connection between a wall and a slab made of prefabricated concrete elements. Direct bearing of the prefabricated elements on the wall. Additional reinforced concrete above the concrete elements.

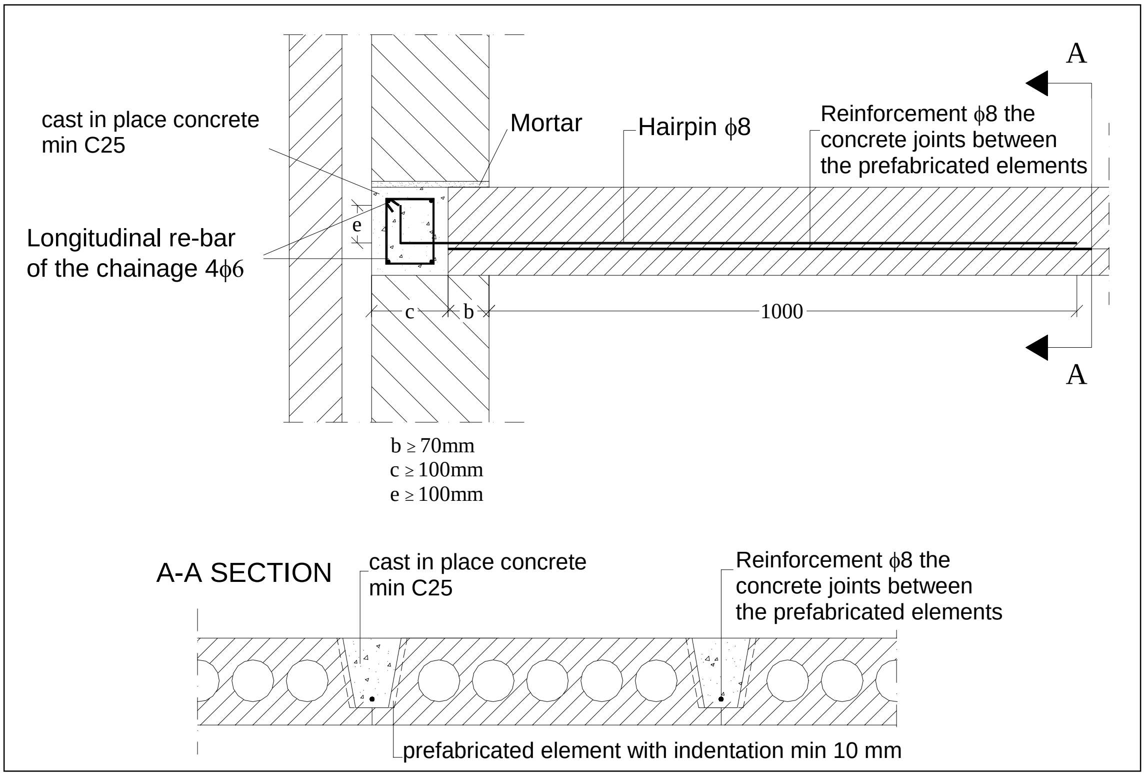


Figure 39. Connection between a wall and a slab made of prefabricated concrete elements. Direct bearing of the prefabricated elements on the wall. No reinforced concrete above the concrete elements.

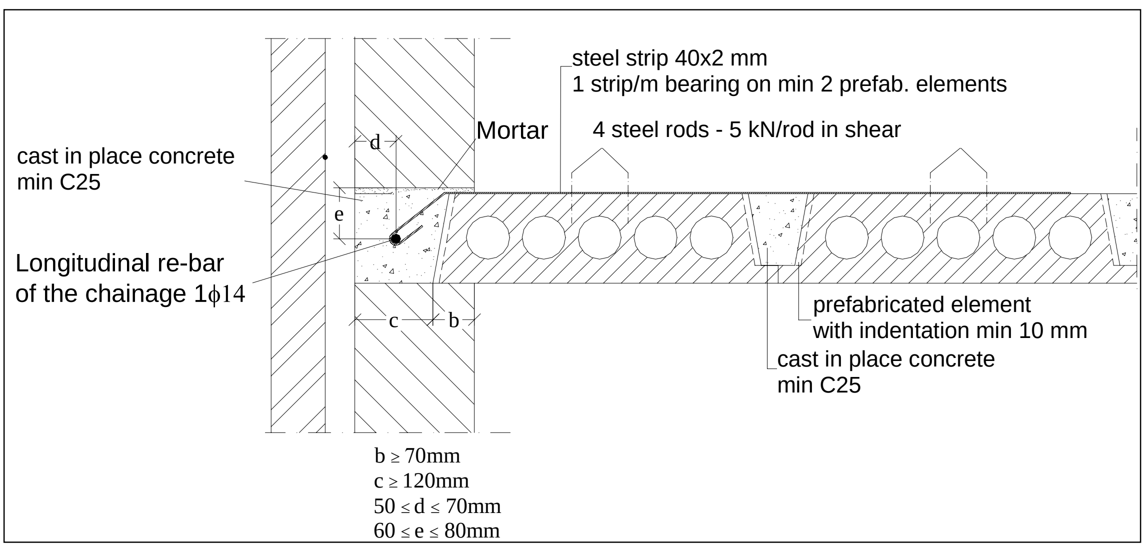


Figure 40. Connection between a wall and a slab made of prefabricated concrete elements. Direct bearing of the prefabricated elements on the wall. No reinforced concrete above the concrete elements.

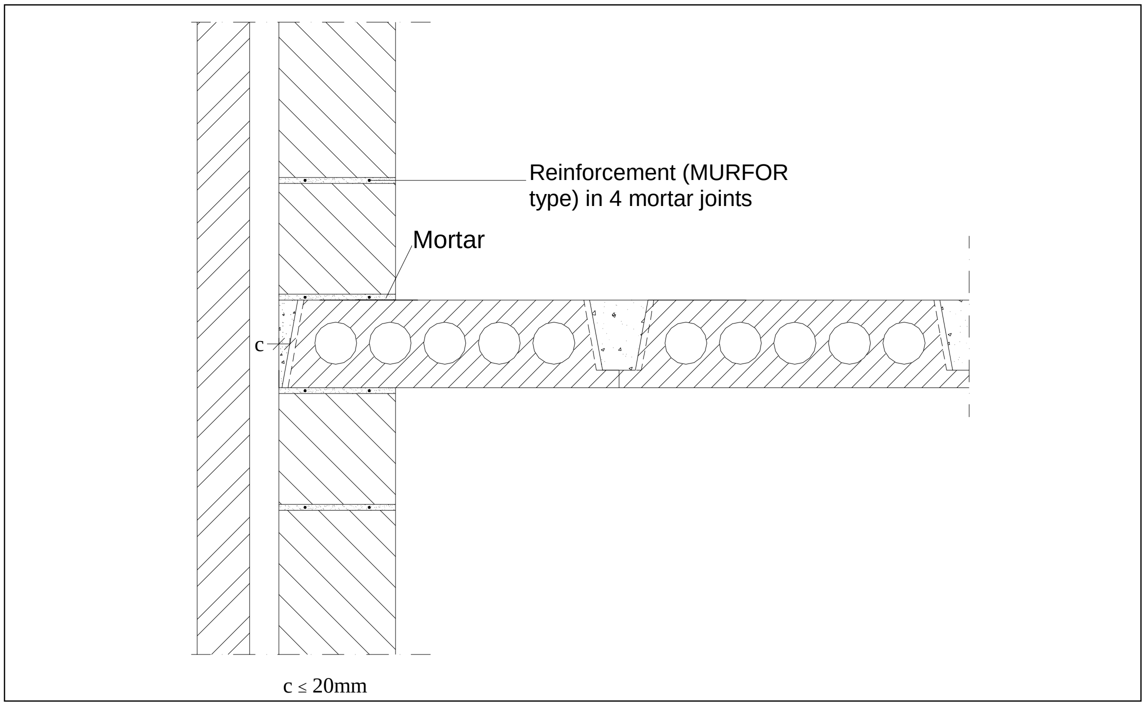


Figure 41. Connection between a wall and a slab made of prefabricated concrete elements. Direct bearing of the prefabricated elements on the wall. No reinforced concrete above the concrete elements. Chainage realised with MURFOR reinforcement.

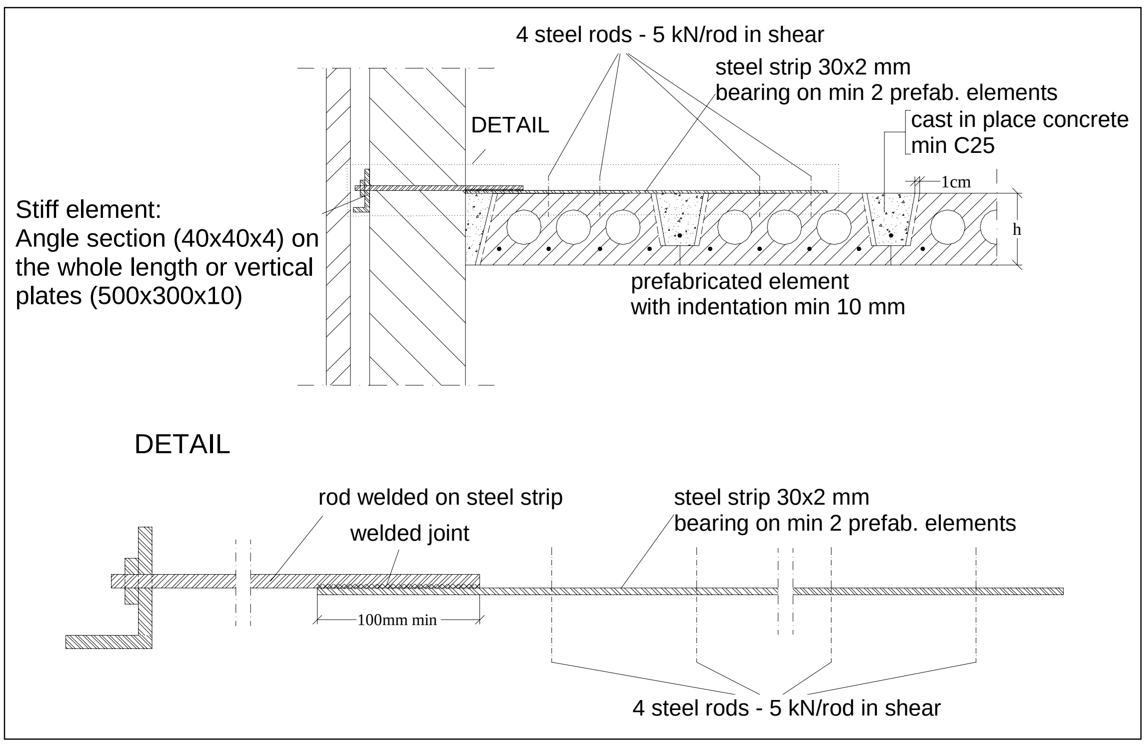


Figure 42. Connection between a wall and a slab made of prefabricated concrete elements. No lateral vertical bearing of the prefabricated elements on the wall. No reinforced concrete above the concrete elements.

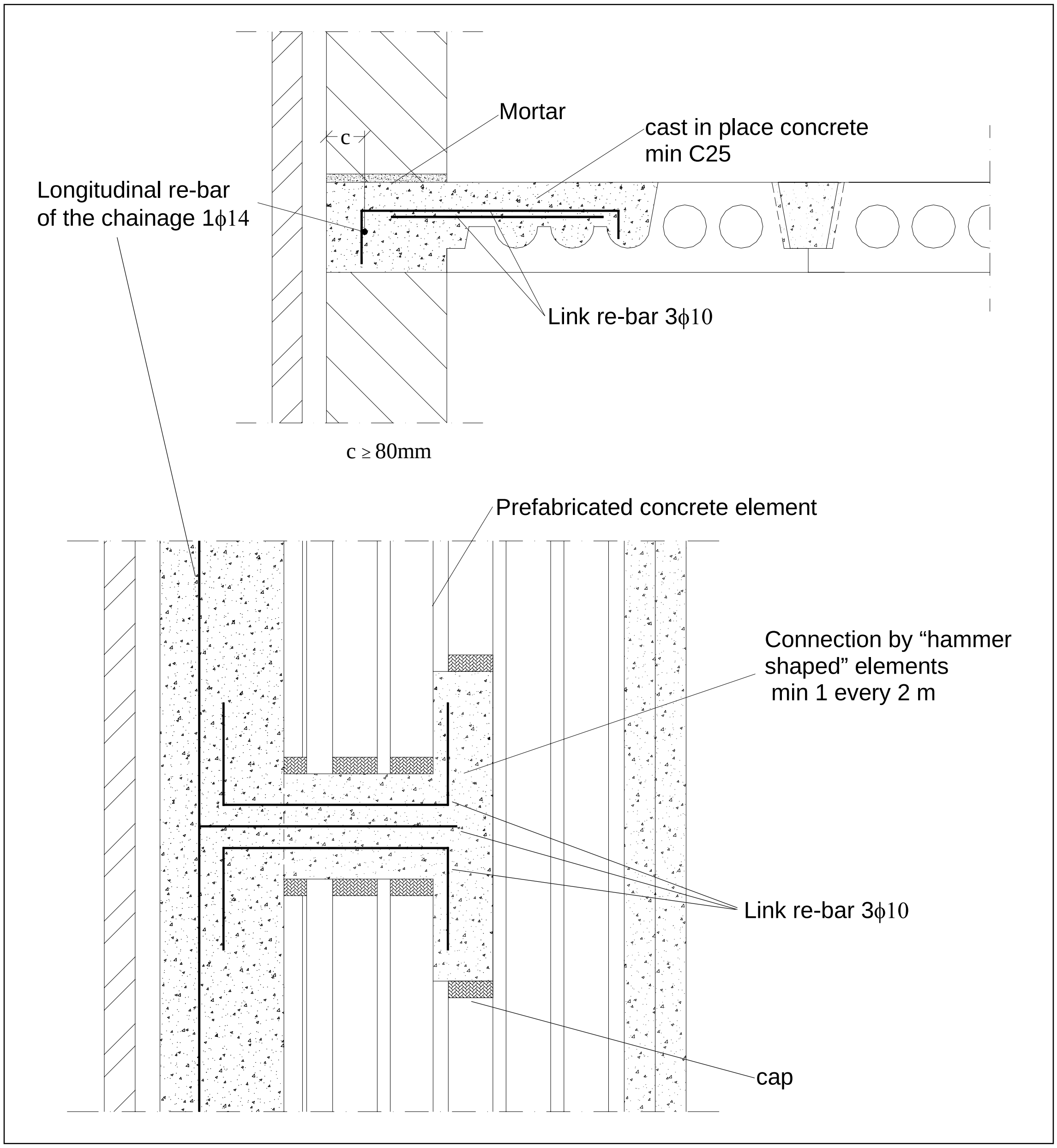


Figure 43. Connection between a wall and a slab made of prefabricated concrete elements. No lateral vertical bearing of the prefabricated elements on the wall (but possible bearing contact). No reinforced concrete above the concrete elements. Specific hammer shaped element in the prefabricated element.

<u>Links between timber floors and walls. Stiffening of timber floors</u>

The objective of the constructional arrangements is to ensure a transfer of forces between floor and wall of 20 kN/m and to ensure an acceptable stiffness of the floor. The Figures 44 to 46 propose some specific details to connect the wall and the main timber beams. Figures 47 and 48 give solutions to transfer tension in the direction of the cross beams (also present but not drawn).

The stiffening of the timber floors is ensured by plywood panels nailed on the main beams and on the cross beams (see Figures 44 and 45).

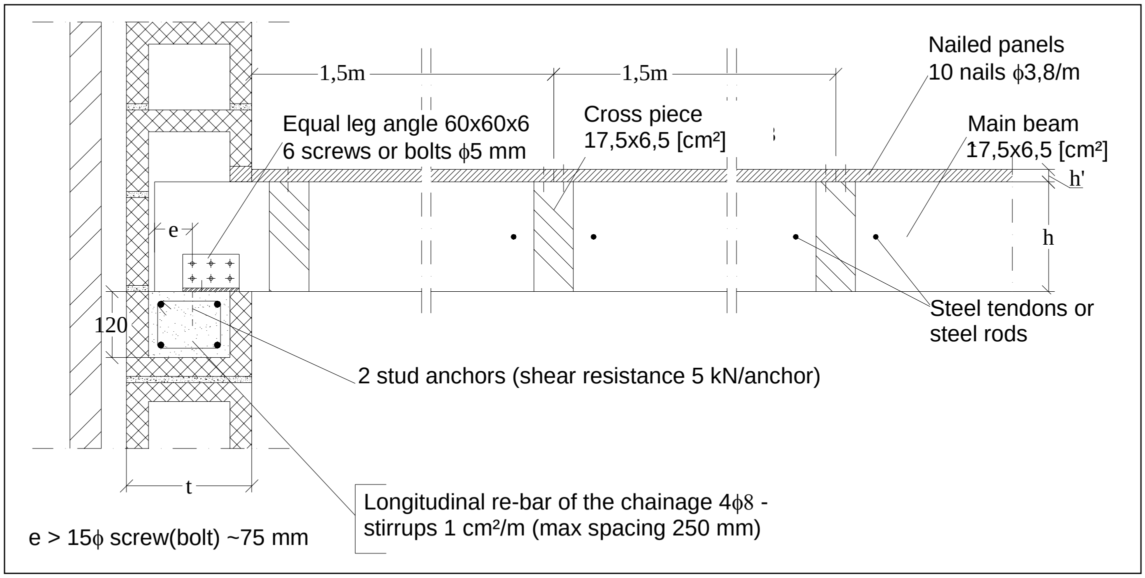


Figure 44. Connection by anchorage of the timber beams on the wall. Stiffening of the floor by nailed panels.

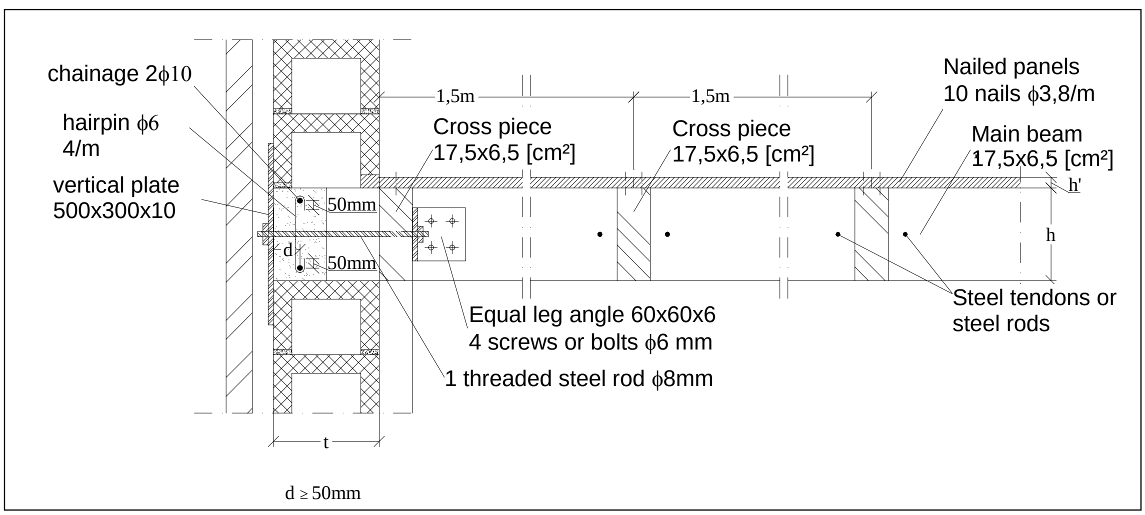


Figure 45. Connection between the timber beams and wall by a plate at the bottom of the wall. Stiffening of the floor by nailed panels.

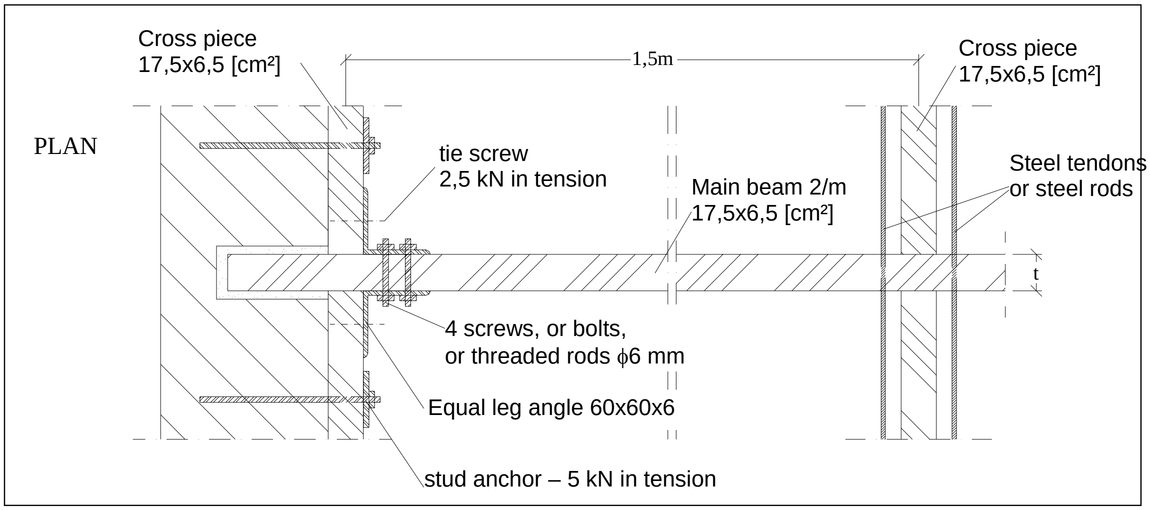


Figure 46. Connection between the timber beams and wall using a timber cross element anchored in the wall by stud anchors.

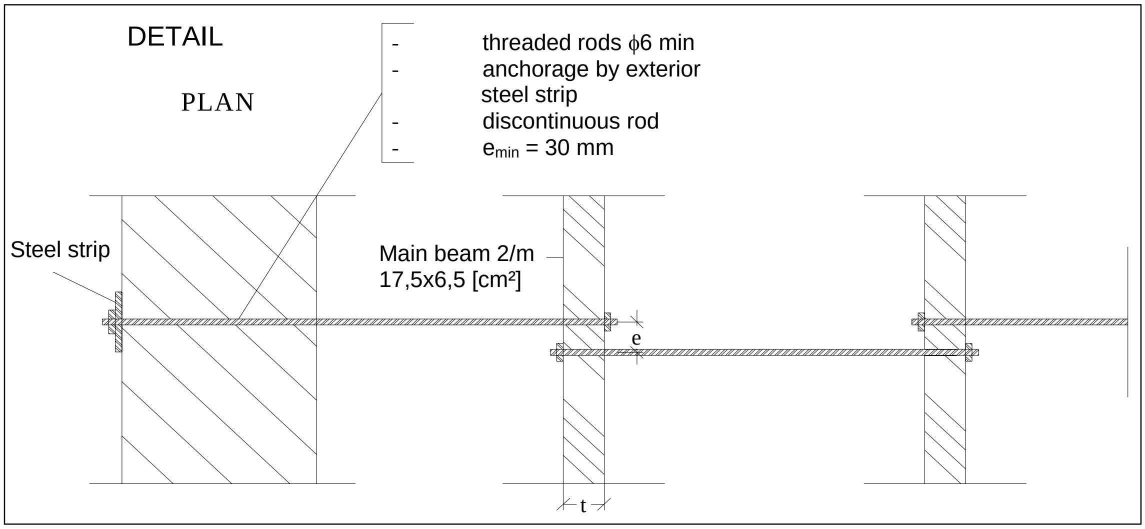


Figure 47. Connection between steel tendon and wall and details of the connections between steel tendons and main timber beams. Steel tendon realized with threaded rods.

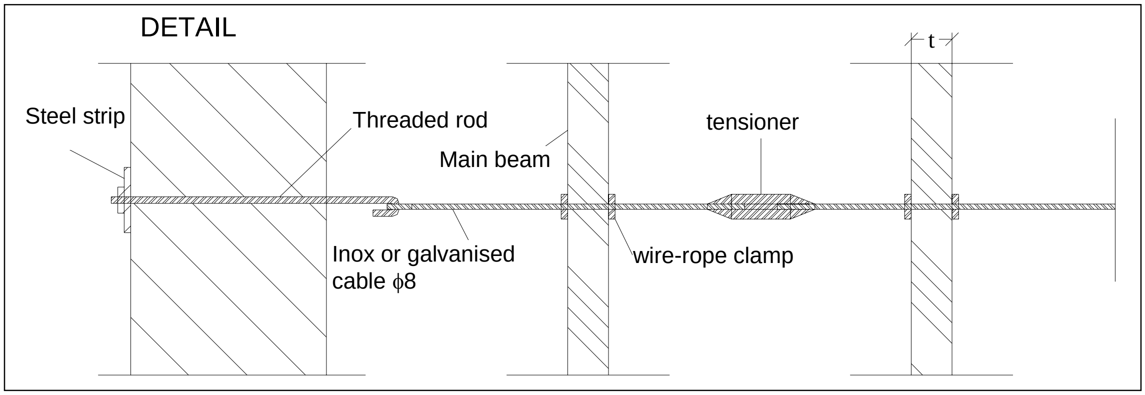


Figure 48. Connection between steel tendon and wall and details of the connections between steel tendons and main timber beams. Steel tendon realized with cables.

<u>Links between roof and walls.</u>

The connections between roof beams and gable wall are designed to withstand 10 kN/m (Figure 51). The connections between roof beams and façade wall are designed to withstand 15 kN/m (Figures 49 and 50).

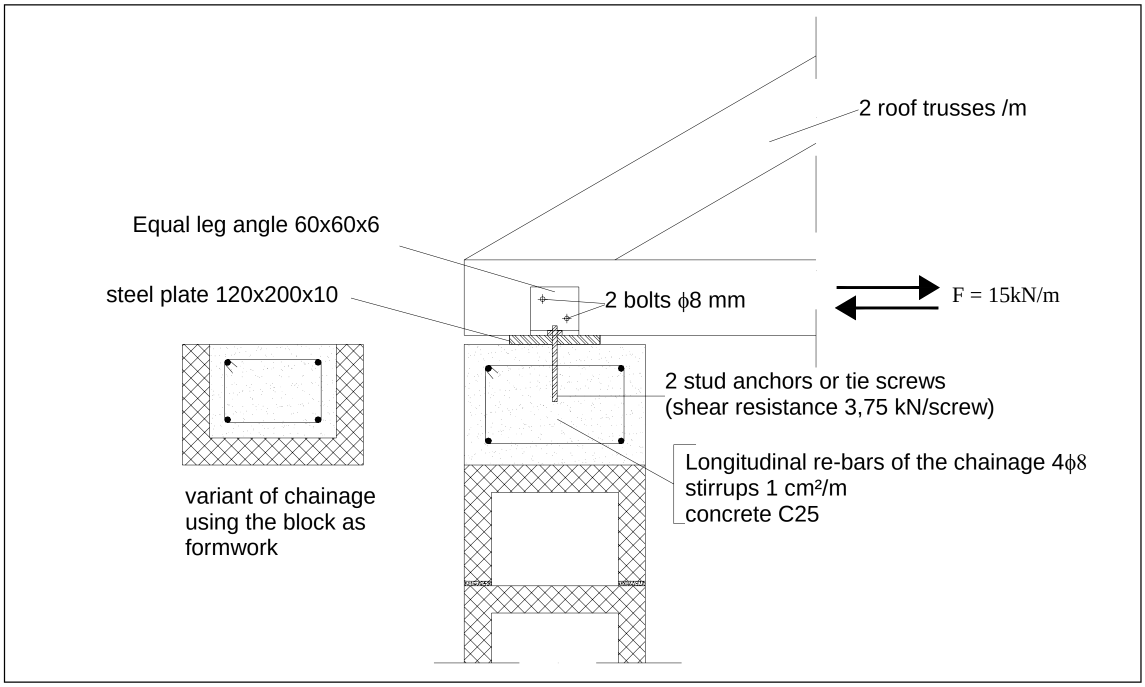


Figure 49. Connection between roof trusses and wall. Anchoring on a reinforced concrete ring beam (chainage).

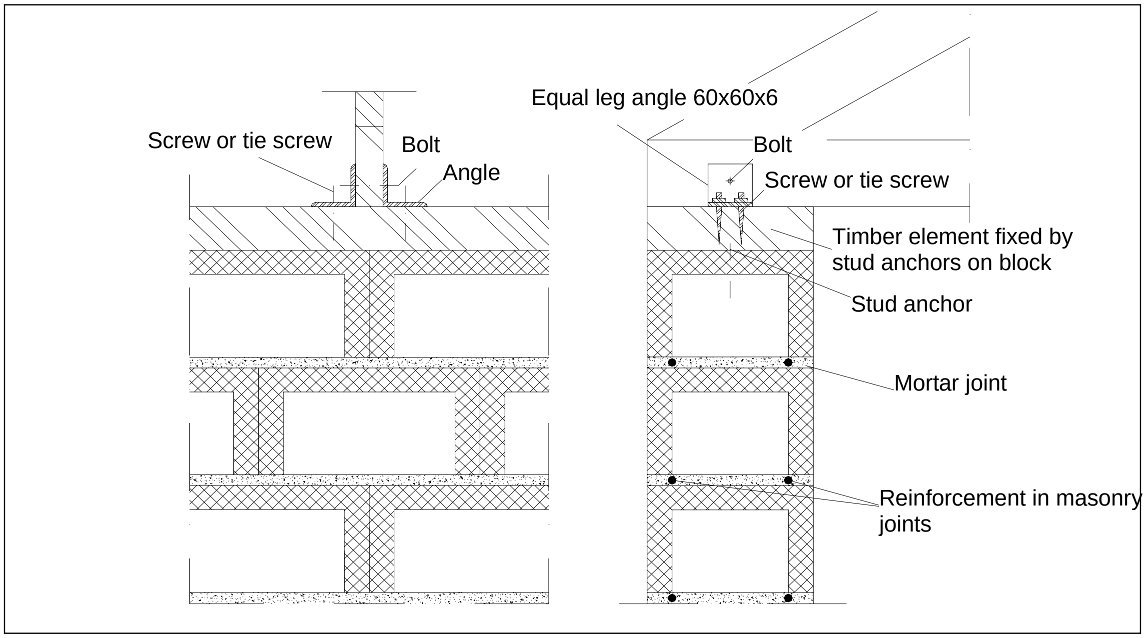


Figure 50. Connection between roof trusses and wall. Anchoring on a timber element fixed by stud anchors on the concrete blocks.

The types of anchoring represented at Figures 49 and 50 should realize two antagonist objectives: transmit the seismic action and allow a displacement (deformation under exterior variable load like snow, wind and thermal or hygrometric dimensional variations). The play between the stud and the hole in the steel plate allow this result.

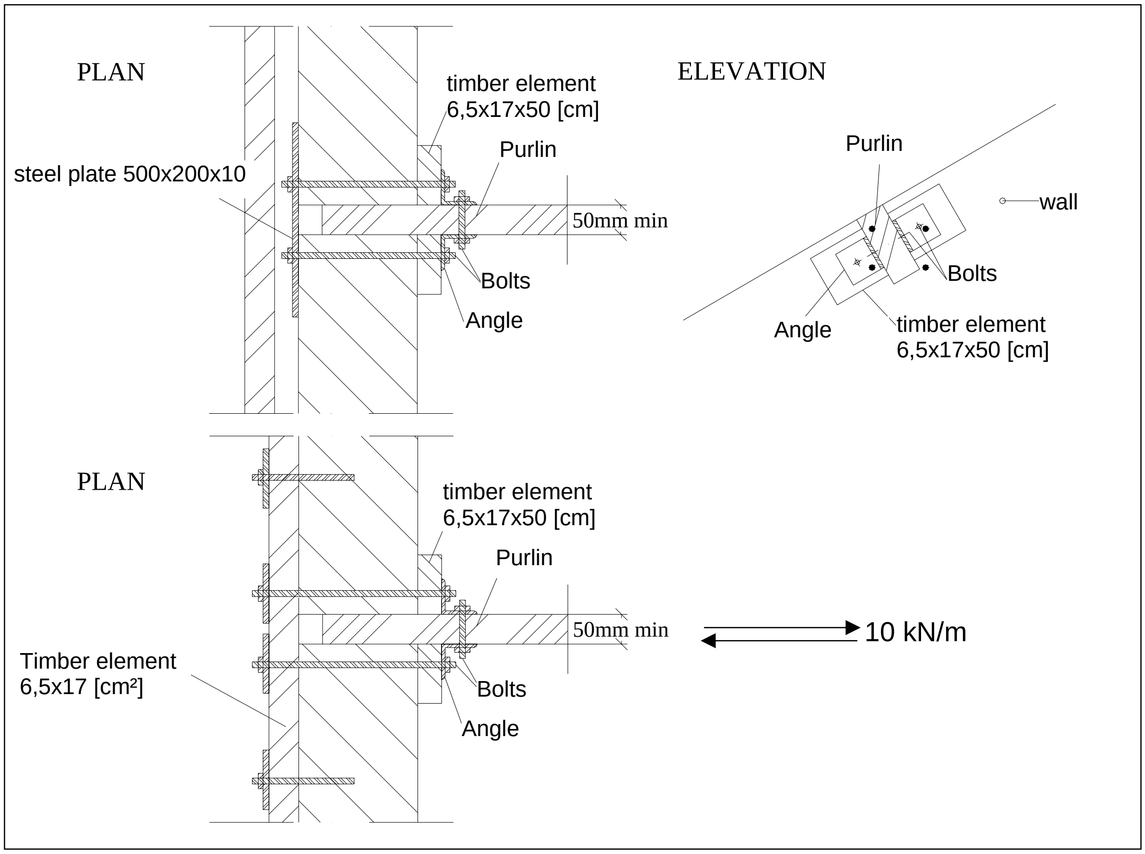


Figure 51. Example of connections between purlin and gable wall.

The different technological possibilities of connections presented in this final report are examples of possible solutions. It is clear that a lot of other technical possibilities exist, which reach the same result: behave correctly under Belgian earthquake. These other valid solutions, to be developed by designers, architects, engineers and construction companies have probably the additional quality to be specific proposals, more adapted to particular enterprise or project contexts. But for builders who would not have time to redesign antiseismic solutions, the presented solutions are good tools directly usable.

4. DISSEMINATION AND DEVELOPMENT

The dissemination of the results has been partly done and will be achieved through different channels.

During the last year of the project, the participants focused their work to already disseminate part of their results:

- A handbook for architects, design offices and civil engineers has been written (Plumier et al. 2003). This handbook contains the specific rules that have to be applied to build simple constructions which are seismic resistant in low seismicity areas and also contains a certain number of construction details easy to be applied in traditional buildings.
- A seminar has been organised on the 6th of February 2003 at the University of Liege, during which the main results of the research, the NAD and the design handbook have been presented to design offices, architects, civil engineers, construction companies and administration. It is intended to organise a second seminar in Brussels, in November 2003.
- The paper "Use of microtremor measurements for assessing site effects in Northern Belgium – interpretation of the observed intensity during the M_S=5.0 June 11 1938 earthquake" by Nguyen F., Van Rompaey G., Teerlinck H., Van Camp M., Jongmans D. and Camelbeeck T. has been accepted for publication in the *Journal of Seismology*;
- A participation to a seminar organised by the French Ministry of Natural Hazards on the 14th March 2003, where the Belgian Science Policy research results have been presented and compared in particular to seismic hazard maps of France and Germany

After the end of the project, the partners will provide their results to the Belgian construction community by:

- The publication of a short text with the main results (in French, Dutch and English), which will be available on the web;
- The publication of a specific design handbook for architect, construction companies and civil engineers, in a professional journal, the "Bulletin du CSTC", CSTC being the BBRI (Belgian Building Research Institute). This channel takes advantage of a very well distributed professional magazine in order to disseminate at best the results of the research
- The publication of the scientific results in international journals.
- A presentation in the GNDT-MAE-SAFERR workshop at Erice (Italy) 26-28 May 2003
- The organisation of two workshops at different locations in Belgium, during which the NAD and the design handbook will be presented to the design offices, architects, public administrations, servicing companies, consultants and construction companies.
- An active contribution to EC8 by transmission of substantiated documents prepared by the research network, using in particularly the channel of Prof. PLUMIER in the EUROCODE-8 Project Team.

It is important to note that a normative document will be written. It will constitute a new release of the National Document of Application for EUROCODE-8, associated with the Euronorm EN version of EUROCODE 8 to be released officially in the summer of 2003.

It will comprise:

- data for the application of the prescriptive part of EUROCODE 8, in particular the design response spectra
- as informative Annex, the set of design details developed in the project, presumably in the BBRI edition version

On another hand, a draft will also be proposed for the parts of EUROCODE-8 that are debated. The writing will be focused on a clear and unequivocal definition of the concepts and the regulations to follow in Belgium and other low seismicity countries. All the partners from the different fields will actively collaborate to this drafting by their participation to the National Committee in charge of seismic problems. This will ensure to obtain a comprehensive document understandable by all the people involved in earthquake engineering.

5. CONCLUSION AND PERSPECTIVE

This study, which concerns the definition of the seismic action to apply to engineering structures and the practical application of building regulations, is the first ever conducted which is based on the real seismic context of Belgium.
It completes the seismic hazard map on the bedrock already realized in the framework of EUROCODE-8 by two fundamental aspects: (a) the definition of elastic response spectra for two different reference earthquakes which should allow to realize a choice for the National Document of Application for EUROCODE-8; and (b) the evidence that regional site effects in the north of the country should be taken into account by a convenient choice of the response spectra. It appears also that the region of the Mons basin should be studied more intensively in the future, because the study suggests that site effects have there disastrous effects on the strong ground motions.

Concerning the practical application of building regulations, a handbook has been written for the architects. This handbook contains the specific rules that have to be applied to build simple constructions which are seismic resistant in low seismicity areas.

The concepts and methodologies used in the framework of this study are not yet taught in Belgian Universities. A conclusion of the project is that now, the “know-how” exists in the Belgian scientific and engineering community. It requires time for the involved researchers to acquire these concepts. The three years duration of the project was sufficient to propose correct and basic information on the seismic action and the construction in low seismicity areas but more time is necessary to consider all the aspects of the investigated problems.

Possible improvements are

1. A better selection of real accelerograms and computation of elastic response spectra using a larger number of data. This should be done in close relationship with colleagues abroad.
2. Including the possible non-linear effects in the “site effects” evaluation
3. More refined numerical modelling of masonry structures: non-linear analysis; finite elements other than shells for simple model of geometrically complex structures.
4. Development of justified simple models for the analysis of highly non regular structures
5. Widening of design recommendations to the building context of other countries.

A step in the direction of the mentioned improvements, in particular those mentioned as 3, 4 and 5 above is presently made thanks to a Belgian Science Policy grant awarded to a Romanian specialist for a research stay at the University of Liege.

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ANNEX 1

SEISMIC HAZARD ON THE BEDROCK IN BELGIUM

The seismic hazard assessment of Belgium in the framework of the EUROCODE-8 has been conducted by the Liege University in cooperation with the Royal Observatory of Belgium (Leynaud et al., 2000). Figure A.1 shows the Peak Ground Acceleration (PGA) geographical variation for a return period of 475 years. The ground movements are considered on the bedrock.

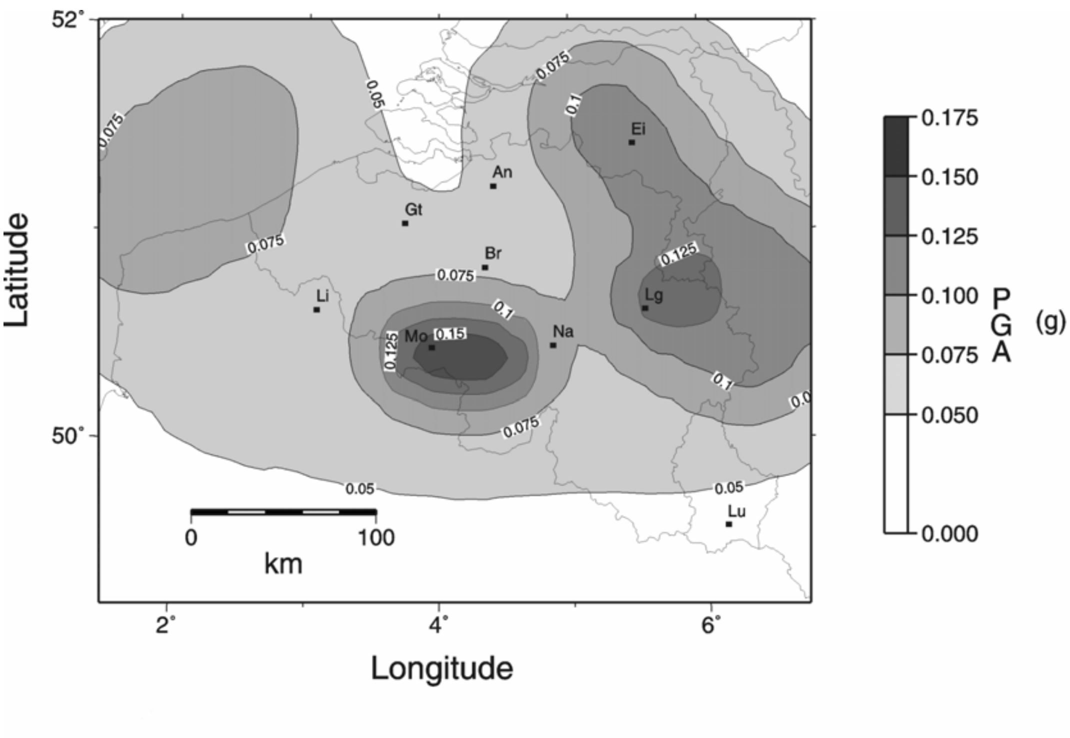

Figure A.1. Peak ground acceleration (PGA) with a 90% probability of non-exceedence in 50 years (return period of 475 years). Ambraseys (1995) attenuation law, including standard deviations, has been considered in the computation.

In the framework of the EC8-DAN technical committee, bringing together scientists, representatives of administrations and engineers to define the Belgian national application document of EUROCODE-8, a seismic zonation has been established from the seismic hazard map. It includes three different zones:

- A zone 0 in which no PGA has to be considered;
- A zone I characterized by a 0,05g PGA;
- A zone II characterized by a 0,1 g PGA.

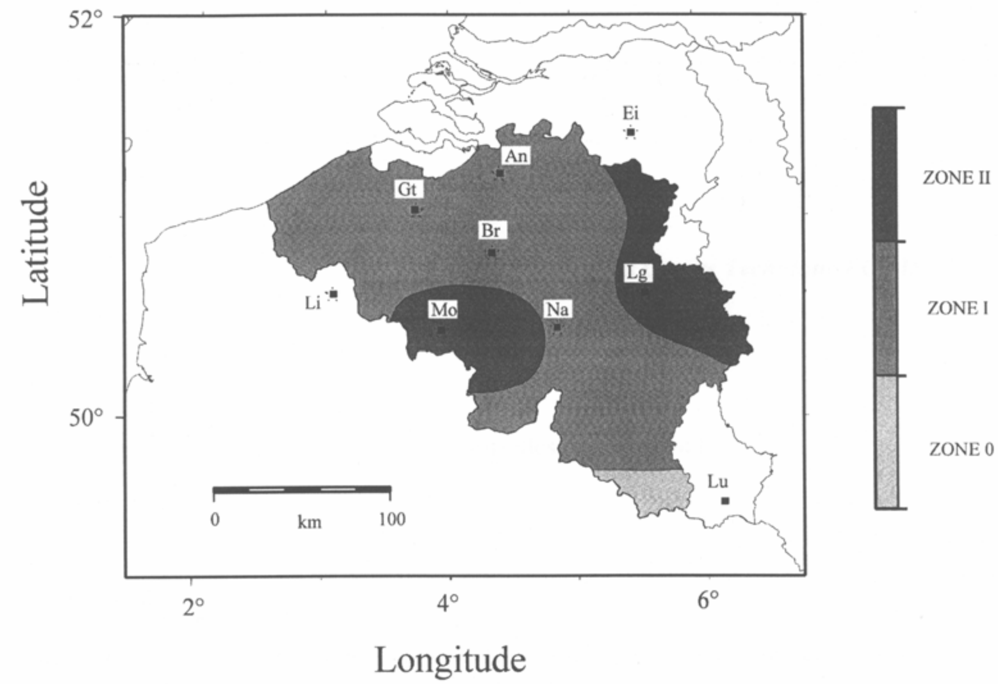

Figure A.2. Seismic zonation of Belgium.

ZONE II – design ground acceleration on the bedrock is 0,1g.

ZONE I – design ground acceleration on the bedrock is 0,05 g.

ZONE 0 – No PGA to consider.

ANNEX 2

RESPONSE SPECTRA IN EUROCODE-8

EUROCODE-8 (may 2001 version) proposes regulation response spectra for different soil conditions (A, B, C, D and E) and for two kinds of earthquakes (*Type 1:* $M_s > 5,5$; *Type 2:* $M_s < 5,5$).
Their shape is also depending on the local effective acceleration value, normally determined with a 10% probability of exceedence in 50 years (return period of 475 years). As an example, the reference response spectra are presented on figure A.3 for the five soil conditions defined in the EUROCODE-8 and for a type 1 earthquake. The considered local effective acceleration is 0,1 g.

The soil classification is resumed in the following table:

Table A.I: Classification of the soils following EUROCODE-8

Soil classes	Lithological description of the soil profiles	Parameters		
		$V_{s,30}$ (m/s)	N_{SPT} (bl/30cm)	c_u[1] (kPa)
A	Rock or other stiff geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
B	Stiff deposits of sand, gravel or overconsolidated clay, at least several tens of m thick, characterized by a gradual increase of the mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of medium dense sand, gravel or medium stiff clays with thickness from several tens to many hundreds of m.	180 – 360	15 - 50	70 - 250
D	Loose cohesionless soil deposits (with or without some soft cohesive layers) or deposits with predominant soft-to-medium stiff cohesive soils.	< 180	< 15	< 70
E	Soils made up of superficial alluvial layer, with a thickness ranging from 5 to 20 m, presenting $V_{s,30}$ values in classes C and D ranges and covering stiffer deposits $V_{s,30}$ > 800 m/s.			
S_1	Deposits consisting or including a level of soft clays or silts with a thickness of at least 10 m with a high plasticity indices (PI > 40) and a high water content.	< 100	–	10 - 20
S_2	Liquefiable soils or soil profiles not included in the classes A-E or S1.			

The S-wave average velocity, $V_{s,30}$, is calculated using the following expression:

[1] Shear resistance, not drained

$$V_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{V_i}} \quad (1)$$

where h_i and V_i are the thickness and the S-wave velocity for the i formations, on a total of N, present in the upper 30 meters. The sites are classifies following this $V_{s,30}$ value if it is available. If not, N_{SPT} will be used.

For sites having soils S1 and S2 characteristics, specific studies are necessary to determine the reference parameters.

Elastic response spectrum

The elastic response spectrum $S_e(T)$ for the reference return period is defined by the following expressions:

$$0 \le T \le T_B : \mathrm{S_e}(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \quad (2)$$

$$T_B \le T \le T_C : \mathrm{S_e}(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \quad (3)$$

$$T_C \le T \le T_D : \mathrm{S_e}(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C}{T}\right] \quad (4)$$

$$T_D \le T \le 4\sec : \mathrm{S_e}(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C T_D}{T^2}\right] \quad (5)$$

where

S_e (T): ordinate of the elastic response spectrum,

T: vibration period of a linear single degree of freedom system,

$a_{g:}$ design ground acceleration for the reference return period,

T_B, T_C, $T_{D:}$ values defining the range of periods defining the curves and the plateau forming the spectrum,

S: soil parameter,

η: damping correction factor.

For the soil classes A, B, C, D and E the values of the parameters S, T_B, T_C and T_D are given for two types of earthquakes:

- An earthquake of type 1 corresponding to a magnitude M_s[2] > 5,5 (table A.II).
- An earthquake of type 2 corresponding to a magnitude M_s < 5,5 (table A.III).

[2] M_s : magnitude of the surface waves

Table A.II: Parameters defining elastic response spectra for an earthquake of type 1

Soil classes	S	T_B	T_C	T_D
A	1,0	0,15	0,4	2,0
B	1,1	0,15	0,5	2,0
C	1,35	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table A.III: Parameters defining elastic response spectra for an earthquake of type 2

Soil classes	S	T_B	T_C	T_D
A	1,0	0,05	0,25	1,2
B	1,2	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

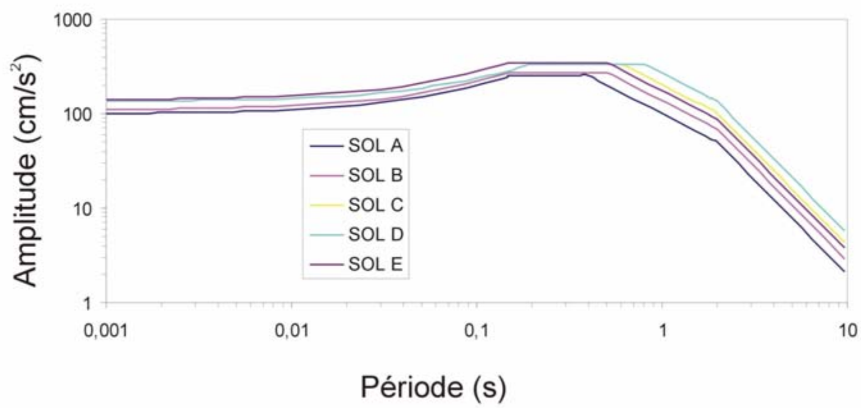

Figure A.3. EUROCODE 8 response spectra corresponding to the different soil classes and for an earthquake of type 1 (PGA=0,1 g).

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SCIENTIFIQUE FEDERALE

rue de la Science 8 ▪ B-1000 BRUXELLES
Tél. 02 238 34 11 ▪ Fax 02 230 59 12
www.belspo.be