

ARTICLE DE RECHERCHE / RESEARCH ARTICLE

French practice for design of embedded walls: history and background, overview of limitations of different models and design approaches

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Abstract – French practice for the design of embedded walls intensively relies on active and passive limit equilibrium analyses, in the tradition of the work by Coulomb, Caquot, Kérisel and Absi, as well as on the more recent method involving spring coefficients, the use of which has become almost systematic, in the continuity of the approaches initiated by Terzaghi and Ménard. This latter method has ever since been improved and refined by intensive geotechnical monitoring, and is of course substantiated, as needed, by the finite element method. The parameters needed to check the serviceability limit states are most often derived from pressuremeter tests, interpreted in accordance with the principles initially established by Ménard, but here again refined by the feedback of monitoring. The ultimate limit states are traditionally checked using global safety factors (approach 2 or now RFA as defined in EC7), the partial factors on the soil resistance parameters (approach 3 or now MFA) being used only for certain situations or types of structures. This tradition, built up on pragmatism, that excludes the unconditional use of a systematic approach and a single method, is based on the belief, every day strengthened by observation and measurement of the behavior of structures, that there is no unique model that would apply to all situations, and that, on the contrary, identification of the limits of existing models is essential. In this lecture, we propose to present the different aspects of this national practice by explaining not only the backgrounds, but also the limits, as well as the way they are taken into account, or at least they should be when this is not the case.

Keywords: geotechnical monitoring / calculation models / linear models / subgrade reaction modulus

Résumé – La pratique française du calcul des écrans : historique et fondements, prise en compte des limites des différents modèles et approches de calcul. La pratique française du calcul des écrans repose très largement sur les équilibres limites de poussée et butée, dans la tradition des travaux de Coulomb, Caquot, Kérisel et Absi, ainsi que sur la méthode, plus récente mais dont l'emploi est devenu quasi systématique, des coefficients de réaction, dans la continuité des approches initiées par Terzaghi et Ménard, et depuis affinées par une pratique intensive du suivi géotechnique, sans oublier bien sûr la méthode des éléments finis, utilisée plutôt en complément en tant que de besoin. Les paramètres nécessaires à la vérification des états limites de service sont le plus souvent issus d'essais pressiométriques, interprétés conformément aux principes initialement établis par Ménard, mais là encore affinés par la pratique du suivi. Les états limites ultimes sont traditionnellement vérifiés à l'aide de coefficients de sécurité globaux (approche 2 ou maintenant RFA au sens de l'EC7), les facteurs partiels sur les paramètres de résistance du sol (approche 3 ou maintenant MFA) n'étant utilisés que dans certaines situations ou certains types d'ouvrage. Cette tradition emprunte de pragmatisme, qui exclut l'usage inconditionnel d'une approche systématique et d'une méthode unique, repose sur la conviction, que renforce chaque jour l'observation et la mesure du comportement des ouvrages, qu'il n'existe pas de modèle universel qui s'appliquerait tel quel à toutes les situations, et qu'il est au contraire essentiel de cerner les limites des modèles existants. Nous nous proposons d'évoquer ci-après les différents aspects de cette pratique nationale en en explicitant non seulement les fondements, mais encore les limites, ainsi que la façon dont elles sont prises en compte, ou dont elles devraient l'être lorsque ce n'est pas encore suffisamment le cas.

Mots clés : suivi géotechnique / modèles de calcul / modèles linéaires / coefficient de réaction

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1 Introduction: evolution of French calculation practice along XXth century, and lessons learnt from geotechnical monitoring

French practice for the design of embedded walls was traditionally based on the Limit Equilibrium Method (LEM) that relies on active and passive earth pressure coefficients, derived from [Coulomb \(1773\)](#) analysis and plasticity theory ([Caquot and Kérisel, 1949](#)), finally resulting in [Kérisel and Absi \(1990\)](#)'s tables.

After the 70's, the LEM was complemented, and progressively replaced by subgrade reaction models, for which efficient softwares were simultaneously developed by R. Fages, for the design of Lyon metro, line 1, and R. Chadeisson, deputy director of Solétanche design office: such softwares made it possible to take account for intermediate values of earth pressure, that apply on the wall when displacements can no longer be considered as high enough to mobilize plastic pressure all along the wall (see [Fig. 1](#)).

Values of the subgrade reaction coefficient, k , were initially derived from Terzaghi empirical proposals ([Terzaghi, 1955](#)), and from Ménard's theory based on pressuremeter tests, calibrated by model tests essentially performed on shallow foundations ([Ménard, 1960](#); [Bourdon and Ménard, 1965](#)).

Unfortunately, both approaches resulted in very low values of k , and accordingly in unrealistically high values of wall displacements, so that further improvements were needed, based on 2 complementary approaches:

- use of finite element models;
- use of geotechnical monitoring, in order to complete Ménard's theory and accordingly provide more realistic values of k , especially in the case of retaining structures ([Bustamante and Gouvenot, 1978](#); [Schmitt, 1984, 1995, 1998, 2009](#)).

Finally, use of geotechnical monitoring became more and more frequent, and is nowadays practically systematic for significant projects.

The reason is that this is obviously the most efficient way to improve both traditional and numerical models, and provide realistic values of soil parameters.

Moreover, the development of deep excavations in urban environments necessitates a strict control of displacements, and a more and more frequent use of the Observational Method ([Holtz et al., 1985](#); [Allagnat, 2005](#); [Lavissee et al., 2007](#); [Schlosser and Schmitt, 2007](#)).

Large databases derived from such intense practice of geotechnical monitoring resulted in the important conclusion that no calculation model may be considered as perfect, which was already obvious, and, on the other hand, that most of them may be considered as valid, provided they are used within their validity range.

This is indeed an important issue, because validity limits of calculation models are scarcely known or described, although one may easily understand that the actual safety factor is significantly reduced when a calculation method is used too close to its limits.

Indeed, geotechnical monitoring is the only way to provide more knowledge about these limits: will it one day be possible to derive model factors that would be apt to account, not for the

validity of a calculation model, but for the compliance of specific project conditions with the validity frontiers of the model employed?

This article tries to provide examples and derive tendencies, based on French experience of about 60 years of geotechnical monitoring practice.

2 Limitations relative to design approaches

Calculation models obviously need to provide results as representative as possible of the actual soil-structure behaviour.

The same cannot be said of design approaches, that employ these models for ULS design, introducing global or partial safety factors the aim of which is not to simulate an actual behaviour in SLS conditions, but to impose a required distance between SLS and failure, with respect to conventional failure mechanisms.

The issue consists in setting a “distance” that would be high enough to satisfy standard safety requirements, but low enough not to result in extra expenses, which is sometimes not so easy to ensure.

Intense discussions about this difficult topic started in the 80's with emergence of Eurocodes that revealed significant differences between European practices.

Indeed, all of these practices are valid, since they have been successfully employed during several decades, and it cannot be evidenced that some countries have developed either unsafe or cost-inefficient practices of geotechnical design!

But, it is now clear that none of these approaches, the validity of which have been demonstrated in relation with specific geotechnical conditions and structural design practices, can be generalized.

For instance, French traditional practice refers to global safety factors, which consists in:

- evaluating global soil resistance with respect to specific failure mechanisms, such as global failure along a critical sliding surface (overall stability as per EC7), or local failure of a retaining structure by lack of passive earth pressure (rotational resistance as per EC7);
- and then specifying safety factors as global ratios between these resistances and exerted actions in SLS conditions, typically in the range of 1.5 to 2.

This was afterwards considered as consistent with approach 2, or more recently RFA, using EC7 successive terminologies, that roughly apply partial factors equal to 1.35 on geotechnical actions, in line with ULS structural calculations, and partial factors varying between 1.1 and 1.4 on geotechnical resistances.

On the other hand, other countries apply partial factors to soil resistance parameters instead of global resistances: they typically impose a 1.25 reduction on $\tan\phi$ and c , in line with approach 3, or more recently MFA, according to EC7 terminologies.

Partial factors, completed by model factors as necessary to facilitate extension of design approaches to different countries, have finally been specified in order to ensure that different philosophies do not result in significant discrepancies in geotechnical design.

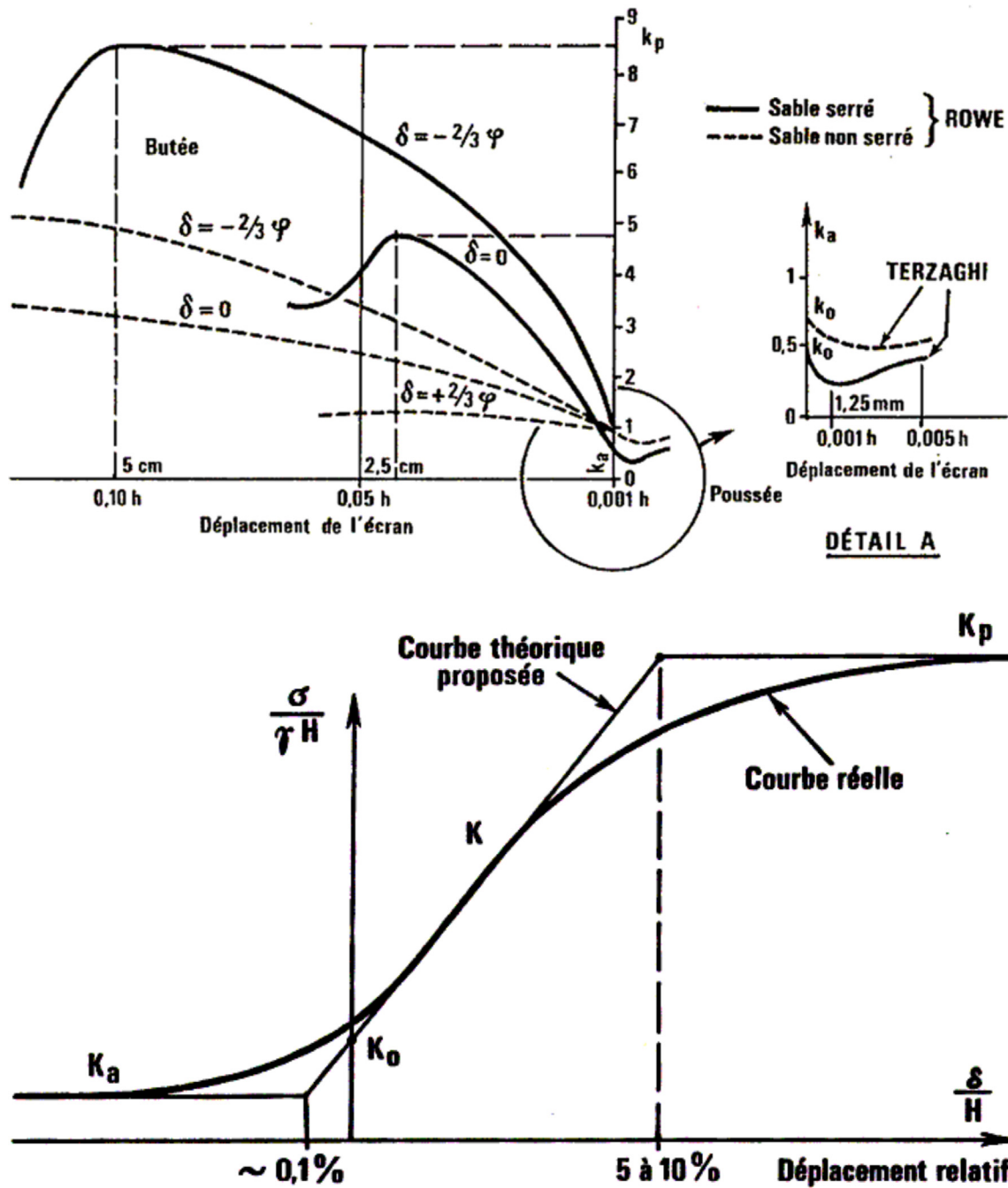


Fig. 1. Synthesis and simplification of Terzaghi and Rowe experiments, after Schmitt (1984).

Fig. 1. Synthèse et schématisation des expériences de Terzaghi et Rowe, d'après Schmitt (1984).

But, this is no longer true when one considers their effects on structural design.

As a matter of fact, although a 1.25 factor may seem a low value compared with 1.5 or 2, consequences of MFA on structural design of retaining structures can be very significant, since such approach simultaneously decreases design passive pressure and increases design earth pressure applied to the structure.

This clearly appears in Figure 2, extract of Baudoin and Frank (2008), where a comparison is made with traditional

ULS approaches of structural calculations, that roughly consist in increasing SLS solicitations by a global factor equal to 1.35.

In practice, it appears that structural calculations developed in accordance with MFA traditionally consider structural plastic hinges that significantly redistribute geotechnical pressures and finally lead to more realistic designs in the case of steel structures, such as sheet pile walls.

But, such approach is not so easy to apply in the case of thick concrete structures such as diaphragm walls: in that case it is, on the contrary, of the utmost importance not to unduly

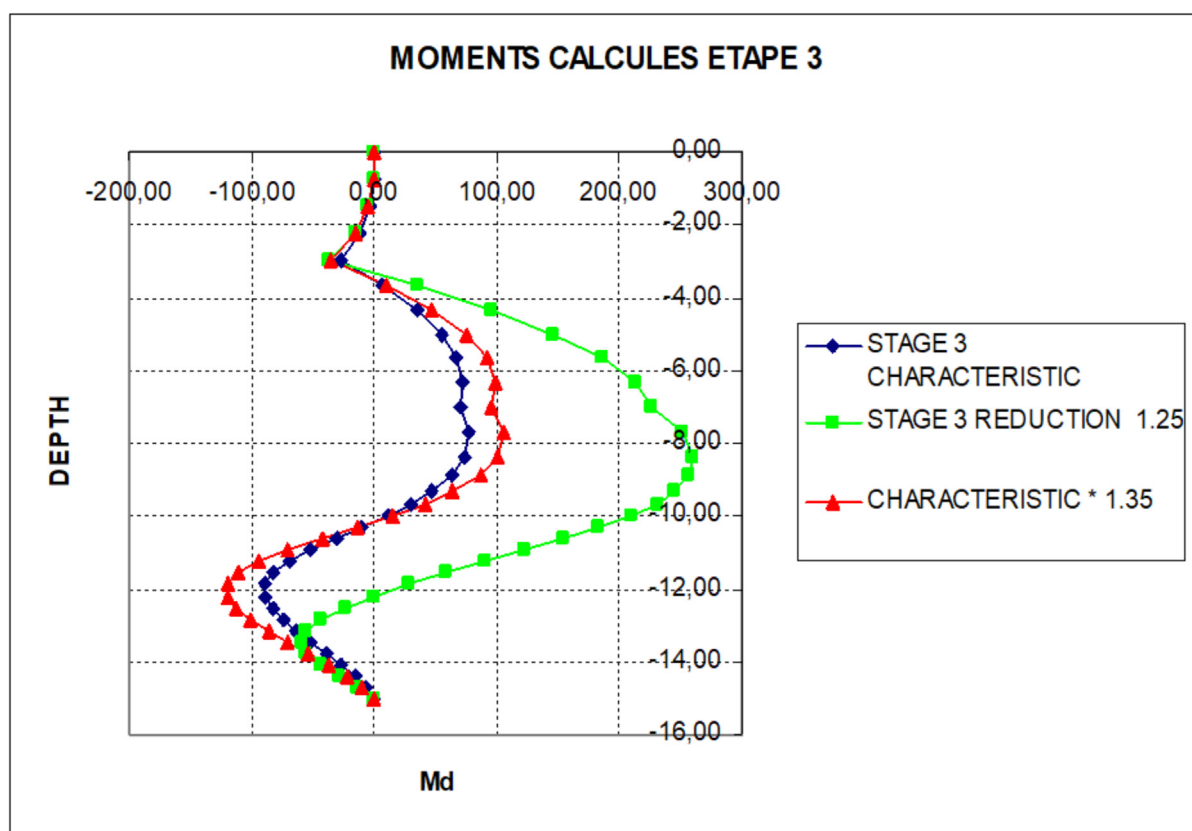
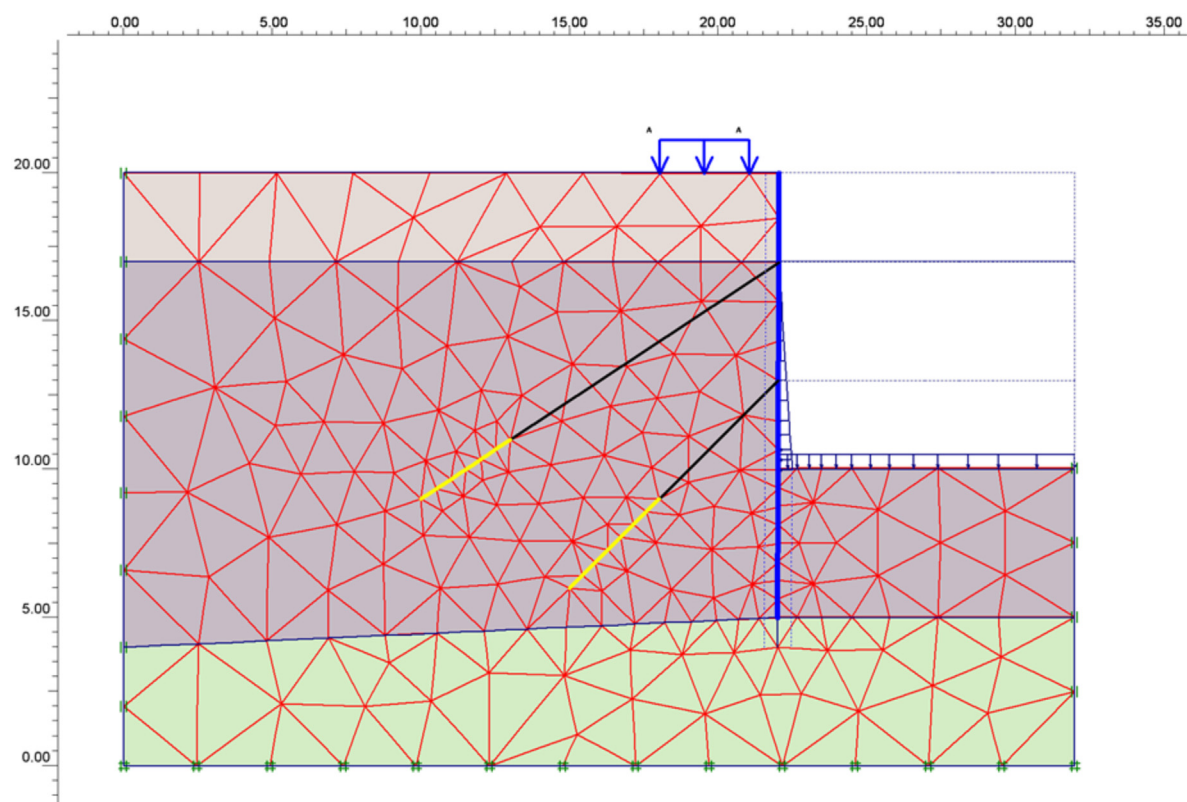


Fig. 2. Ultimate bending moments derived from MFA, after [Baudoin and Frank \(2008\)](#).

Fig. 2. Moments ultimes déduits de l'approche MFA, d'après [Baudoin and Frank \(2008\)](#).

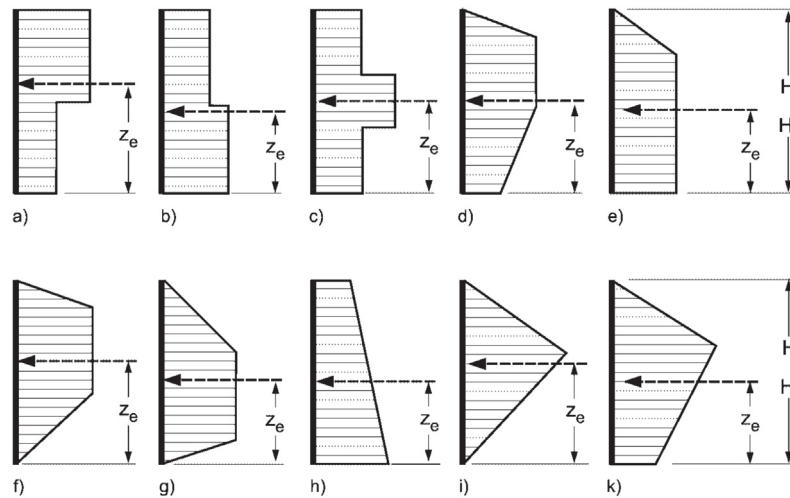


Fig. 3. Examples of alternative earth pressure diagrams, after EAB (2013).

Fig. 3. Exemples de diagrammes de poussée forfaitaires, d'après EAB (2013).

increase design solicitations and finally result in unrealistic densities of steel reinforcement, with potentially detrimental consequences in terms of concrete placement as well as cost efficiency.

Finally, although MFA is generally considered as a relevant way to deal with overall failure mechanisms, for which safety is essentially provided by soil resistance itself, attempts to generalize it to other limit states finally evidenced severe limitations in terms of cost-efficiency. This is especially the case for those mechanisms that significantly interfere with structural resistance, such as local failure of retaining structures, and for specific categories of structures, such as reinforced concrete ones, for which it is neither current nor easy, with respect to existing standards, to consider plastic hinges in ULS calculations.

Financial consequences of MFA compared with RFA are even more difficult to assess as they depend on the calculation model itself.

For instance, using conventional Bishop models to check overall stability with respect to deep failure mechanisms has generally no consequence on the design of structures that may be encountered within the slope, as long as they do not interfere with the sliding surface: as a matter of fact only consequences of soil resistance decrease along the deep failure surface itself need to be considered.

But this is no longer true if the same design approach is used in combination with numerical models, such as FEM, that consider a general decrease of soil resistances within all the soil mass.

Nevertheless, if imposing a universal design approach that would apply in all countries, for all kinds of structures, situations, limit states, and in combination with all calculation models, is not realistic, a combined use of design approaches based on a better knowledge of their limitations might indeed be the base for a common language, in line with French traditional practice and recent proposals made at European level, as developed in Section 3.

3 Limitations relative to overall displacements

Validity limits of Limit Equilibrium Models, that exclusively rely on conventional earth pressure coefficients, are known to be related to displacements of the retaining structure, that, as reminded in Figure 1, need to be high enough to mobilize plastic soil behaviour on both sides of the wall.

Some countries have extended these limits by replacing traditional earth pressure coefficients by conventional earth pressure diagrams, such as shown in Figure 3 (EAB, 2013), or developing subgrade reaction models to describe intermediate soil behaviour between at rest and plastic pressures: this is clearly the choice that has been made in France.

But limits relative to overall displacements themselves are not explicitly addressed: limits of a few 0.1% earthside and 1% on the excavated side, necessary to mobilize full plastic behaviour, are generally stated in terms of relative displacements of the wall itself (see Fig. 1), implicitly accepting that overall displacements of the soil mass behind may not be strictly equal to zero, without saying any more about their possible limits.

Meanwhile, Figure 4, showing the evolution of failure surfaces with the inclination of an existing slope, based on PARIS software developed by Solétanche-Bachy in order to extend Coulomb's methodology to non planar sliding surfaces (Boussinesq, 1876; Caquot and Kérisel, 1949; Salençon, 1974; Dodel *et al.*, 2002), suggests that such limits exist.

As a matter of fact, this figure clearly shows a progressive evolution from a conventional earth pressure plastic edge, only generated by the wall displacement, towards an intrinsic slope instability, for the prevention of which the wall more and more seems to play a significant part: in the last situations, the overall displacement of the soil mass may not be neglected, since it clearly tends to be higher than the local wall displacement itself (in some extreme situations (Blondeau and Virollet, 1976) "flowing" of soil above the wall could even be observed).

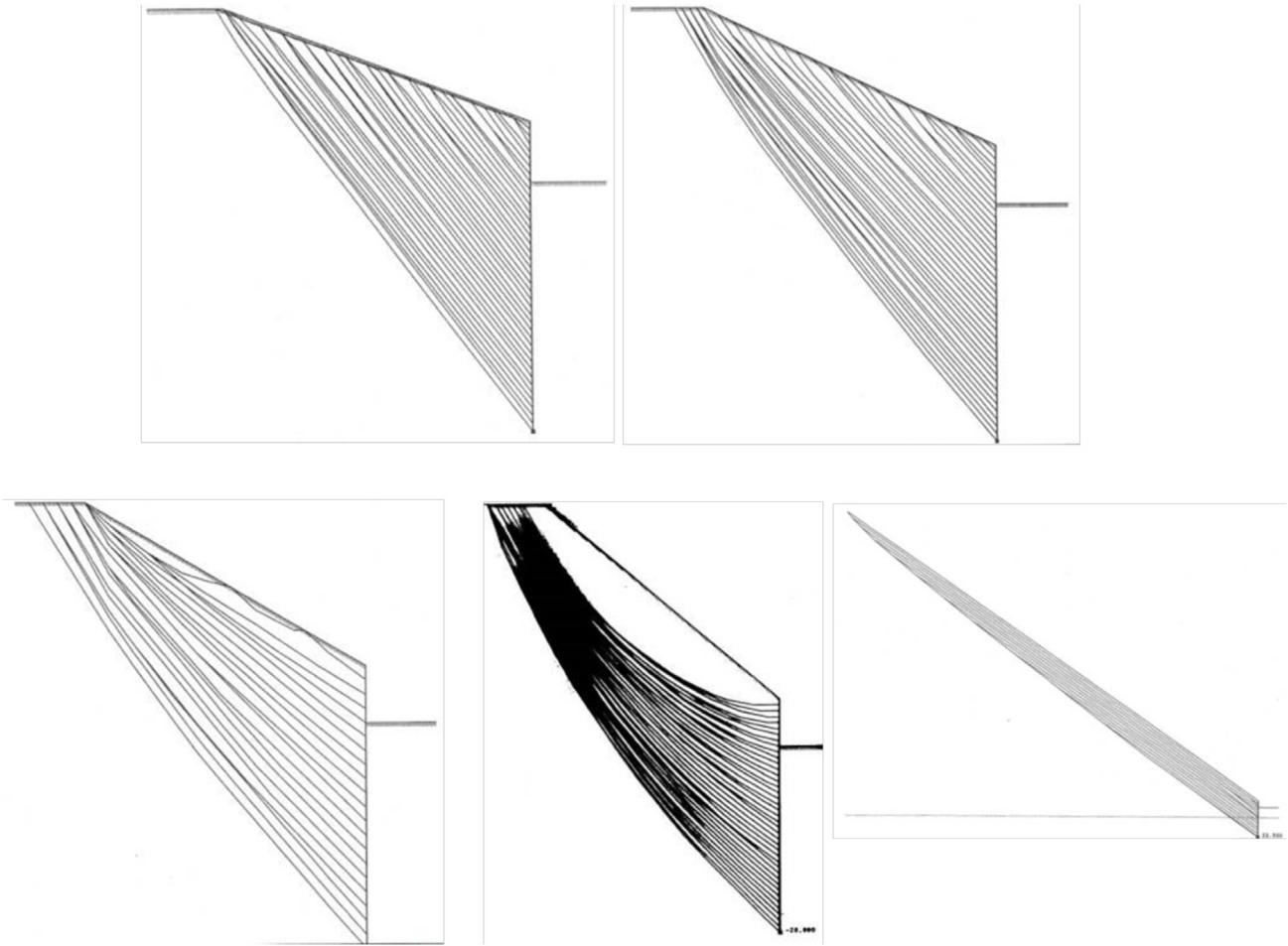


Fig. 4. Effect of increasing slope inclination, after [Dodel et al. \(2002\)](#).

Fig. 4. Effet de l'augmentation de la pente du terrain, d'après [Dodel et al. \(2002\)](#).

This means that earth pressure can no longer be described by a plastic active state, but rather by a passive one, potentially involving earth pressure coefficients about 10 times higher than expected.

Clearly, conventional methods are not able to quantify, and not even predict such evolution, since they are not able to consider overall displacements, and thus implicitly assume that such displacements are really negligible: this is indeed an important limit in their validity area that should be more explicitly clarified, as it is intended to be done in the next issue of EC7.

This may explain why it has always been common practice, in countries such as France that make an intense use of these conventional models, to impose large safety factors (typically in the range of 1.5 as already mentioned, in line with RFA): such safety factors implicitly serve to guarantee as well that overall displacements can actually be neglected, and that the design situation lies far enough from the validity limits of the calculation model.

Should not this be verified, for instance within existing slopes, for which safety factors may be significantly lower, the occurrence of higher earth pressures should then be considered, using either more appropriate models, or higher safety

factors for structural design, as implicitly done when MFA is used, unless a reinforcement of the existing slope may be undertaken.

Indeed, it is very difficult to explicit, in the context of Eurocodes, that the safety factor relative to a specific ULS (*i.e.*, overall stability), may determine the choice of the calculation model, or even of the design approach itself, that needs to be employed to check another ULS (*i.e.*, structural resistance of a retaining structure), but unfortunately this is reality!

This reinforces the conclusion of [Section 2](#), relative to the existence of frontiers between design approaches, expressed either in terms of cost efficiency for MFA or overall displacements for RFA.

This may also explain the choice that has been made in France to preferably apply RFA for the design of embedded walls in standard conditions, and MFA for the design of nailed structures, for which reinforcements are placed during excavation itself, resulting in potentially lower safety factors and higher displacements in temporary situations: the consequence is that the retaining structure itself actually needs to be designed as a reinforcement, involving calculation methods and approaches explicitly derived from overall stability considerations.

4 Limitations relative to subgrade reaction models

Although this model has been successfully employed for several decades and all over the world, many specialists express the conviction that its validity area is really tiny, which may easily be understood.

As a matter of fact, the model relies on the well-known theory of beams resting on elastic supports that cannot seriously be considered as representative of soil mass behavior, since it is a simplification of linear elasticity theory, whilst soil behaviour is known to be neither elastic nor linear!

Moreover it neglects the effect of shear stresses within or between different soil layers, by considering them as a series of independent springs, which obviously is unrealistic.

Although this theory had been initiated by respectable figures of soil mechanics, such as Westergaard (1926), Terzaghi (1955), Ménard (1960) and Bourdon and Ménard (1965), it was really a surprise to discover, in the early 1980's, when trying to validate recently developed software and provide realistic values of subgrade reaction coefficients based on geotechnical monitoring, as already mentioned in Section 1, that the model itself was finally quite representative of actual behaviour of embedded walls in standard conditions.

But selected values of k , derived from back-analysis, needed to be significantly higher than initially proposed by above-mentioned authors (Bustamante and Gouvenot, 1978; Schmitt, 1984).

Figure 5 only provides an example among the numerous results of topographic and inclinometer measurements that may nowadays be found in geotechnical literature (Londez *et al.*, 1997; Millotte and Fonty, 2022; Schmitt, 1984, 1995, 1998, 2009).

Of course, such factual but unexpected observation requires theoretical explanations, which will be the object of Section 5.2, as well as a strict definition of frontiers of the validity area, that obviously exist, even if they are more distant than initially assumed.

4.1 Validity limits of the model earthside

As far as earth pressure earthside is concerned, the representativity of the subgrade reaction model and uncertainties relative to the evaluation of k may not be considered as a major issue, due to the low values of displacements that are necessary to mobilize active pressure, as already mentioned.

Indeed, limitations of the model are essentially due to limits of conventional earth pressure theories themselves.

4.1.1 Limitations relative to overall displacements and settlements

First of all, no information can be expected in terms of overall displacements, that need to be considered as negligible, as already mentioned in Section 3, as well as settlements, the evaluation of which necessitates the complementary use of either numerical or empirical models (Holtz *et al.*, 1985; Phienweij *et al.*, 1998; Bazin and Schmitt, 2001; Nguyen, 2003; Marten, 2005; Sayavong, 2008).

4.1.2 Limitations relative to non-uniform loads

An important limitation also comes from the fact that conventional earth pressure theories do not consider the effect of non-uniform loads acting behind the wall, for which specific methods had to be proposed and their validation discussed (Boussinesq, 1885; Terzaghi, 1943; Houy, 1976; Bazin *et al.*, 1994; Dodel *et al.*, 2002).

Current situations, such as small berms or existing buildings, may nowadays be treated using these specific methods, described by normative documents (AFNOR, 2009), but the validity areas of which still need to be more accurately specified.

4.1.3 Limitations relative to underground structures

Interaction with existing underground structures is more difficult to deal with, and each situation requires a specific analysis.

Numerical models should sometimes be employed.

4.1.4 Limitations relative to anchored walls

A frequent situation is related to anchored or multi-anchored walls, that support the retaining structure by transferring the load to the soil mass behind (Schlosser, 2007), which makes it necessary to check that the anchor force is applied far enough from the retaining structure not to interfere with it.

This is generally checked using the Kranz method (Kranz, 1953; AFNOR, 2009), that consists in checking the ratio between the maximum anchor force that can be exerted without increasing earth pressures considered for the design of the wall, and the applied anchor force, derived from its calculation in SLS conditions.

Indeed this ratio, typically required not to exceed 1.5, should not be considered as a safety factor, since it does not necessarily relates to a specific ULS, but rather reflects an indicator of the project situation with respect to validity limits of earth pressure models.

For instance, Figure 6 (Hanauer and Utter, 2006; Schlosser and Schmitt, 2007) provides an example of a project for which, due to site constraints, upper anchors are much too short to meet the Kranz condition when deep excavation levels are met, so that they interfere with the retained structure.

But this interference does not result in any ULS since it is considered in the building sequence that is a top-down in the lower levels, which enables upper slabs to be simultaneously constructed and accordingly contribute to stabilization of the retaining wall.

Clearly, such complex interaction could not be considered using only conventional models, and the design relied upon numerical models that automatically consider all the interactions.

Indeed, although not referring to the same mechanism, the Kranz coefficient is used in a similar way as the global safety factor relative to overall stability, which, as mentioned in Section 3, may also be used as an indicator checking validity limits of the conventional models employed for the design of the structure itself.

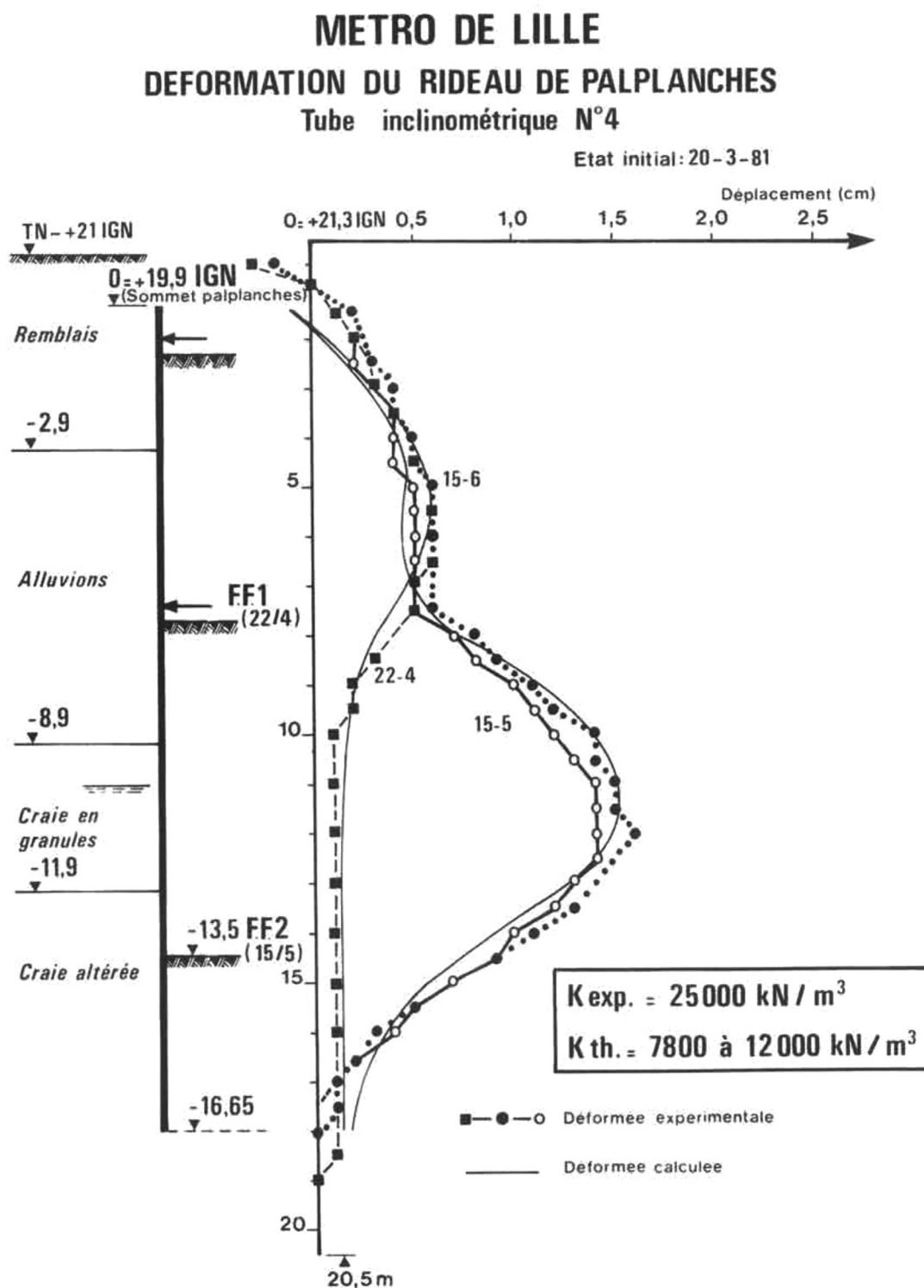


Fig. 5. Comparison between calculated and measured deflections, after Schmitt (1984).

Fig. 5. Comparaison de déformées calculées et mesurées, d'après Schmitt (1984).

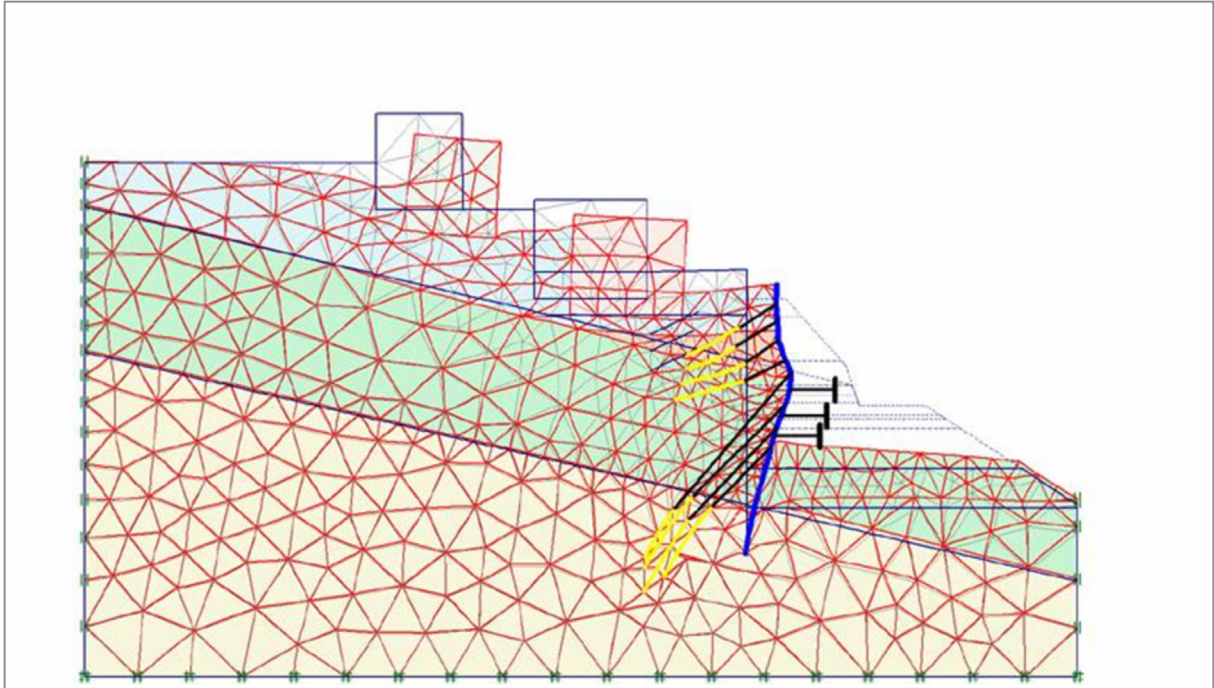


Fig. 6. Interaction between retained structure and anchors, after [Schlosser and Schmitt \(2007\)](#).
Fig. 6. Interaction entre soutènements et ancres, d'après [Schlosser and Schmitt \(2007\)](#).

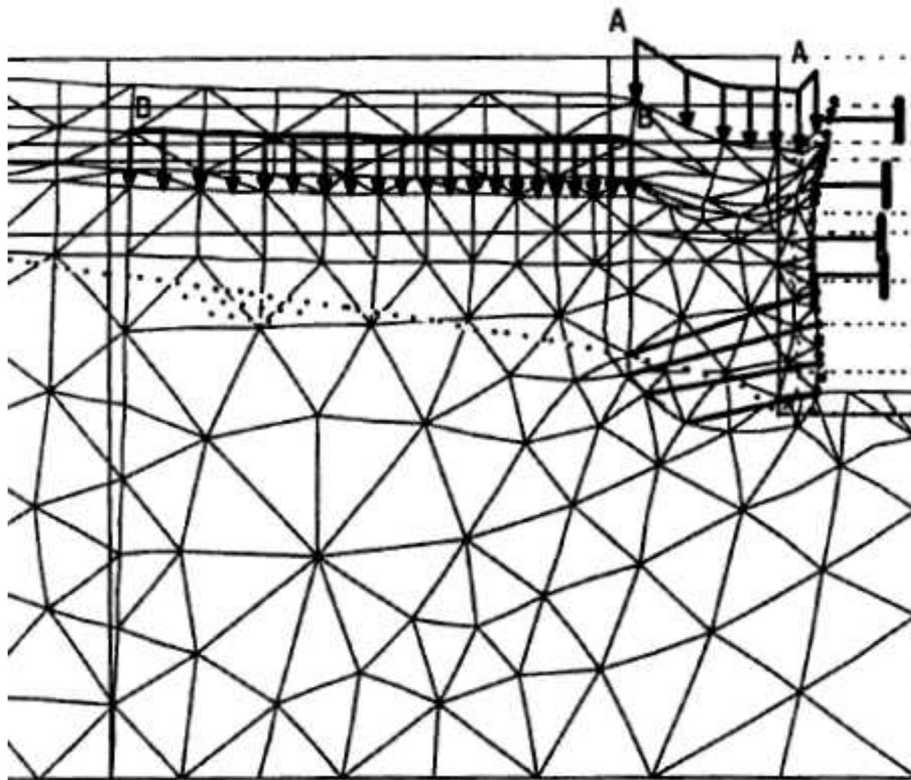


Fig. 7. Example of load transfer from soft to rigid supports, after [Schmitt \(2009\)](#).
Fig. 7. Report de charge des appuis souples vers les appuis rigides, d'après [Schmitt \(2009\)](#).

Finally, such mechanisms (overall displacements [Sect. 4.1.1] or interference with anchors [Sect. 4.1.4]) may nowadays be safely dealt with, since the validity limits of conventional models used to design the retaining structure may be appreciated in each situation thanks to indicators.

But other mechanisms exist for which indicators still need to be proposed.

4.1.5 Limitations relative to arching effects

For instance, Figure 7 (Schmitt, 2009) provides an example of a composite earth retaining structure, made of a berlin wall supported by rigid struts in the upper part, and by a nailed structure supporting weathered rock in the lower part.

Finite elements calculations show that the displacement of the lower structure, that is very flexible compared with the stiffness of the upper supports, is quite significant.

This results in some kind of overall displacement, the consequence of which is an arching effect, and finally a load transfer from weathered rock to struts, that is obviously not considered if the design is made using conventional models, that do not account for any kind of overall displacement.

This may also occur, even for homogeneous structures, in case passive supports with significantly different rigidities are used (*i.e.*, combination of rigid struts and passive anchors), or when high spans exist between successive support levels, more especially when combined with a low flexural rigidity of the wall itself.

As a matter of fact, in such circumstances, horizontal displacements within the soil behind may significantly differ between rigid supports levels, that restrain displacements, and other locations: this may potentially result in formation of vaults and local mobilization of passive earth pressure, that can be modelled neither by active earth pressure coefficients, as mentioned in Section 3, nor by subgrade reaction coefficients, that cannot consider the mobilization of shear stresses and redistribution effects.

In practice, comparison with finite elements models (for instance Daktera, 2020) shows that such circumstances may result in an increase of support reactions, without any significant effect in terms of bending moments, since higher soil pressures are strictly located behind rigid supports.

It could also be shown (Kazmierczak, 1996; Kastner, 2018) that such discrepancies disappear when supports are pre-stressed: this may be easily explained by the fact that mobilization of passive earth pressure is in such a case mainly the consequence of the wall displacement towards the soil, rather than of the soil displacement towards the wall, the former being correctly modelled by subgrade reaction calculations.

It must also be emphasized that no example derived from geotechnical monitoring of the wall itself could be provided so far, which can be explained, maybe by the general preponderance of water rather than earth pressures, but also by the fact that increases in support reactions, without significant consequences in soil deflections and bending moments themselves as already mentioned, cannot be evidenced by inclinometers.

Reactions in passive struts themselves are scarcely measured, except for Lyon or Grand Paris metro that confirmed

such increases in a few locations, as already mentioned (Kastner, 2018; Daktera, 2020).

Clearly, more investigations would be necessary in order to better quantify specific limits of conventional models, that should be considered, as already mentioned, in the case of rigid passive supports, and more especially contrast between support rigidities at different levels, or high spans between supports in case of low rigidity of the retaining wall itself.

In such circumstances, numerical models or empirical diagrams (as shown in Fig. 2) should be envisaged, even if no significant damage due to arching effects was ever experienced so far.

Indeed, large safety factors in case of passive struts are generally used in order to deal with other effects that are generally difficult to predict, such as temperature variations and support conditions, which may suggest, of course not to neglect the possible occurrence of arching effects, but rather to use additional model factors directly applied to strut forces, taking advantage of the fact that bending moments themselves should not be affected.

4.2 Validity limits of the model on the passive side

As shown in Section 4.1, limitations of the model earthside are essentially due to limitations of conventional earth pressure theories themselves.

The importance of assessing representative value of k is much more important on the passive side, due to the large span, in terms of displacements, that needs to be modelled between at rest and passive earth pressures (see Fig. 1).

It may accordingly seem strange that such significant area can be properly modelled by a linear relationship: this was initially explained by strict limitations generally imposed to wall displacements, possibly resulting in an initial tangent soil modulus being more representative of such small displacements.

This could also explain the necessity of using higher values of k than initially proposed by Terzaghi and Ménard, who essentially relied on the observation of foundation settlements that possibly covered a larger range of soil deformation, in agreement with Figure 8 (Atkinson and Sällfors, 1991; Reiffsteck, 2002).

A more reliable explanation will be proposed in Section 5, based on a more general analysis of what may be considered as a “representative” modulus in the specific case of excavation projects.

But before discussing values of k coefficient, it must be emphasized that the validity limits of the model itself are clearly the limits of the well-known theory of beams resting on elastic supports.

4.2.1 Limitations relative to the theory of beams resting on elastic supports

A general criticism consists in observing that assuming a constant ratio between local earth pressure under a foundation and local settlement is not realistic, and not even consistent with the theory of linear elasticity itself.

This clearly demonstrates that the validity range of subgrade reaction models is limited to circumstances for

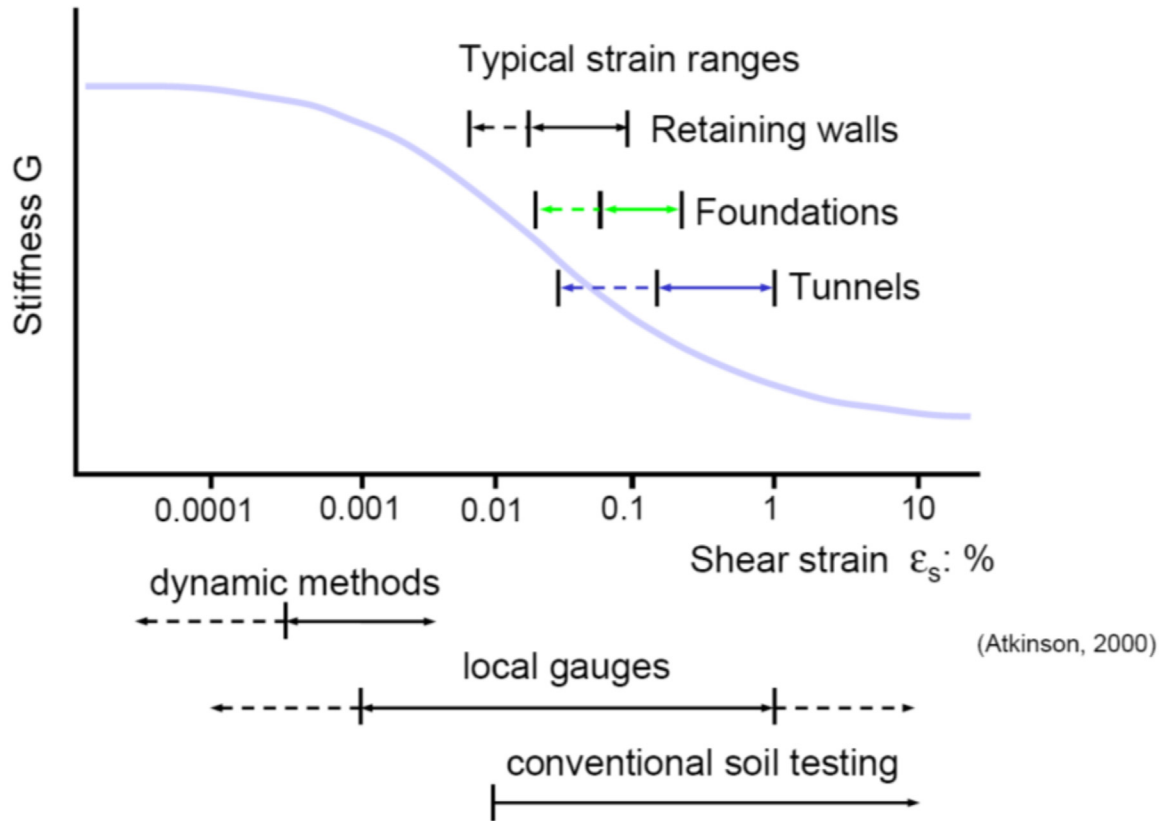


Fig. 8. Dependency of soil stiffness with shear strain, after Reiffsteck (2002).

Fig. 8. Influence des déformations sur la raideur du sol, d'après Reiffsteck (2002).

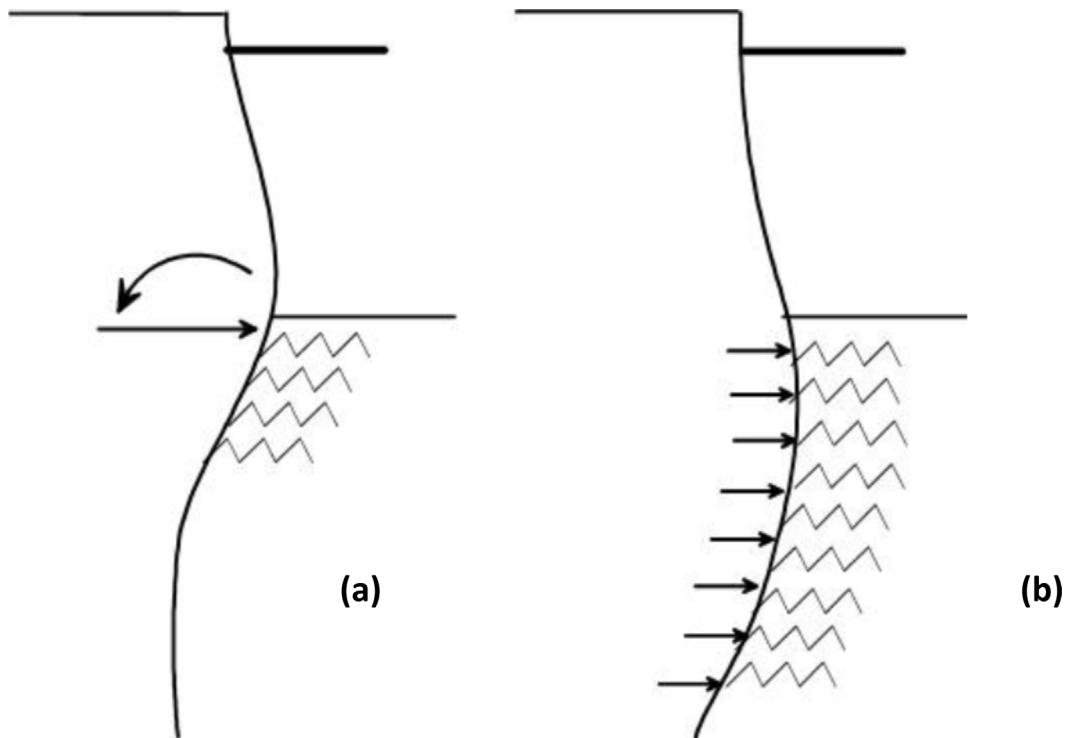


Fig. 9. Comparison between mechanical (a) and hydraulic (b) embedment models, after Schmitt (2009).

Fig. 9. Comparaison entre comportements de fiches mécanique (a) et hydraulique (b), d'après Schmitt (2009).

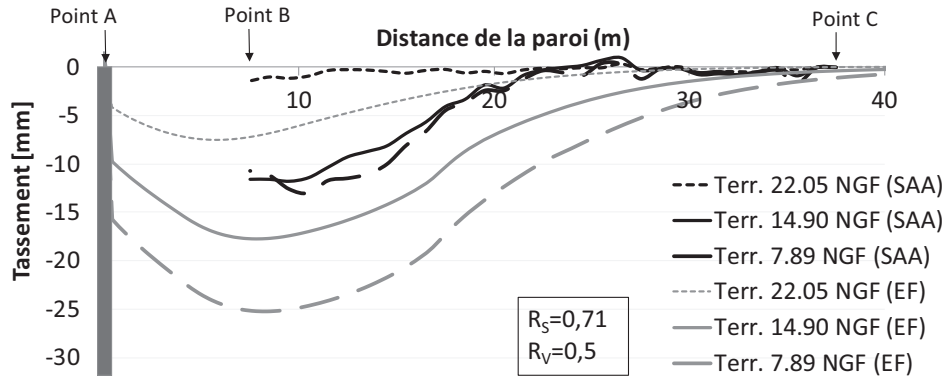


Fig. 10. Comparison between calculated (SAA) and measured (EF) settlements, after [Daktera et al. \(2020\)](#).

Fig. 10. Comparaison entre cuvettes de tassements mesurées (SAA) et calculés (EF), d'après [Daktera et al. \(2020\)](#).

which local variations of stresses are not that important, so that only the ratio between the average pressure and the average displacement over the soil structure interaction area needs to be considered: this should be considered as the actual definition of k coefficient.

As far as the passive earth pressure interaction area is concerned, and more especially considering that the embedded part of the retaining structure essentially serves to transmit the horizontal shear force necessary to stabilize the upper part of the wall, as shown in [Figure 9a](#), experience shows that such assumption may in practice be considered relevant, and that the theory of beams resting on elastic supports is representative.

As a matter of fact, comparison with inclinometer measurements confirmed ([Schmitt, 1998](#)) that the interaction height along which wall displacements may in such a case be considered as representative, say higher than 20% of the maximum displacement, is, as predicted by calculations, in the range of $a = 1.5 l_0$, where $l_0 = (4 EI/k)^{1/4}$ and EI is the bending inertia of the wall.

This clearly validates this simplified theory in conditions described by [Figure 9a](#).

Based on the French database, a representative value of the soil modulus E_s could be derived from the pressuremeter modulus E_M using the empirical relationship $E_s = 3.6 E_M/\alpha$, where α is the "structure coefficient" based on Ménard's theory (as discussed in [Sect. 5.2](#)).

This results in a representative evaluation of k , at least of its order of magnitude, that was proposed by ([Schmitt, 1995](#); [AFNOR, 2009](#)):

$$k \sim E_s/a \sim 3.6 \cdot E_M/(\alpha \cdot a) \sim 2 \cdot (E_M/\alpha)^{4/3}/(EI)^{1/3}. \quad (1)$$

As already mentioned, we will not detail in the present article databases from which these empirical formulae were derived ([Bustamante and Gouvenot, 1978](#); [Londez et al., 1997](#); [Millotte and Fonty, 2022](#); [Schmitt, 1984, 1995, 1998, 2009](#)), but will rather focus on the limits they highlight.

4.2.2 Limitations relative to deep water pressures

As [Figure 9b](#) shows, it may occur that the embedded part of the retaining wall do not serve only to stabilize the upper part, but also to equilibrate deeper geotechnical actions, such as water pressure, when it has been necessary to reach an

impervious layer at depth, or during pumping tests before excavation begins, or in the case of deep quay walls subject to tide effects ([Bazin and Schmitt, 2001](#); [Delattre et al., 2005](#); [Marten, 2005](#); [Sayavong, 2008](#); [Daktera, 2020](#)), that can also result in significant displacements.

In such a case, the interaction height may be significantly higher than predicted by the theory of beams resting on elastic supports.

Even if the general order of magnitude $k \sim E_s/a$ may always be assessed, representative values of a , and even of E_s , may be more difficult to predict, and should be cautiously assessed, considering for instance an extension of the interaction height to the total embedded length, unless more analysis or data are available ([AFNOR, 2009](#)).

4.2.3 Limitations relative to the geometry of the retaining structure

In the case of short or rigid walls compared with soil stiffness, the interaction length implicitly considered in the Schmitt formula (1) may be higher than the wall length itself, which is not realistic.

In such a case, " a " should obviously be reduced in order not to exceed the embedded length of the wall ([AFNOR, 2009](#)).

4.2.4 Limitations relative to the geometry of the excavation

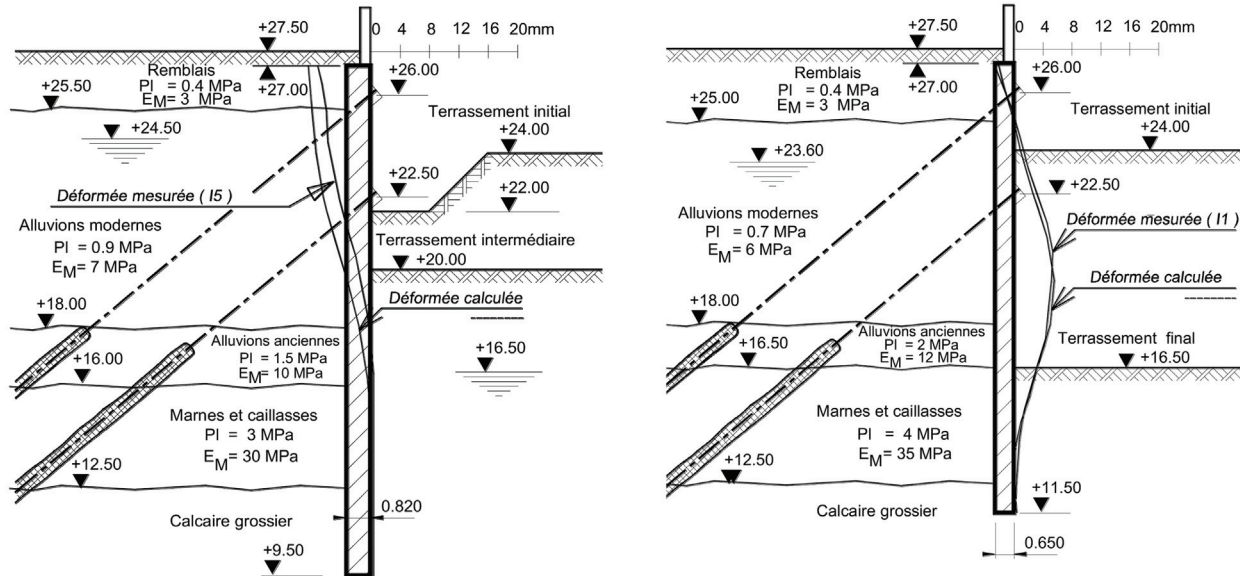
Narrow excavations may result in an interaction between facing retaining walls, and finally higher values of k , that should in such a case be limited by simple consideration of the Hooke's formula ([Nguyen, 2003](#)).

Passive earth pressure coefficients themselves may be underestimated, which may be corrected using model factors based on plasticity theory confirmed by model tests.

4.2.5 Limitations relative to project duration

Since empirical theories and coefficients are essentially based on geotechnical monitoring, as already mentioned, it should be considered that unfortunately such monitoring is generally limited to construction stages, so that they cannot account for potential time effects, such as consolidation or creep.

(a)



(b)

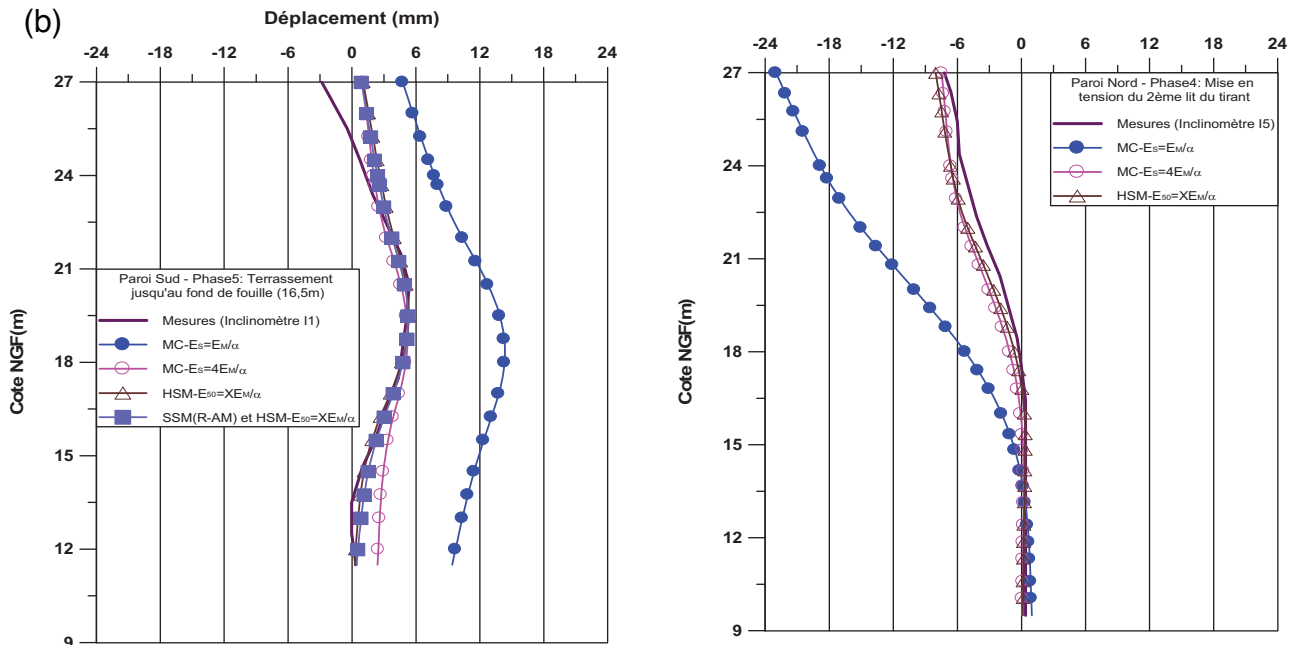


Fig. 11. Comparison between measured and calculated deflections using subgrade reaction (a) and numerical models (b), after Plumelle *et al.* (2005).

Fig. 11. Comparaison entre déformées mesurées et calculées à partir de la méthode du coefficient de réaction (a) et de la méthode des éléments finis (b), d'après Plumelle *et al.* (2005).

A few examples of long term monitoring in silty or clayey layers (Bustamante and Gouvenot, 1978; Londez *et al.*, 1997) evidenced significant increases in local as well as overall displacements.

Hence lower stiffness coefficients should in such a case be used based on measurements of long-term moduli, as long as sufficient monitoring results in permanent conditions are not available to provide more guidance.

Clearly, use of empirical models always need a careful evaluation of project conditions in order to confirm they

effectively lie within validity limits, and in order to derive representative values of soil parameters.

5 Limitations relative to finite elements models

5.1 General limitations

Validity areas of finite elements, and more generally numerical models, are obviously larger than those of

Tassements en phase finale d'excavation

Distance derrière la paroi (m)

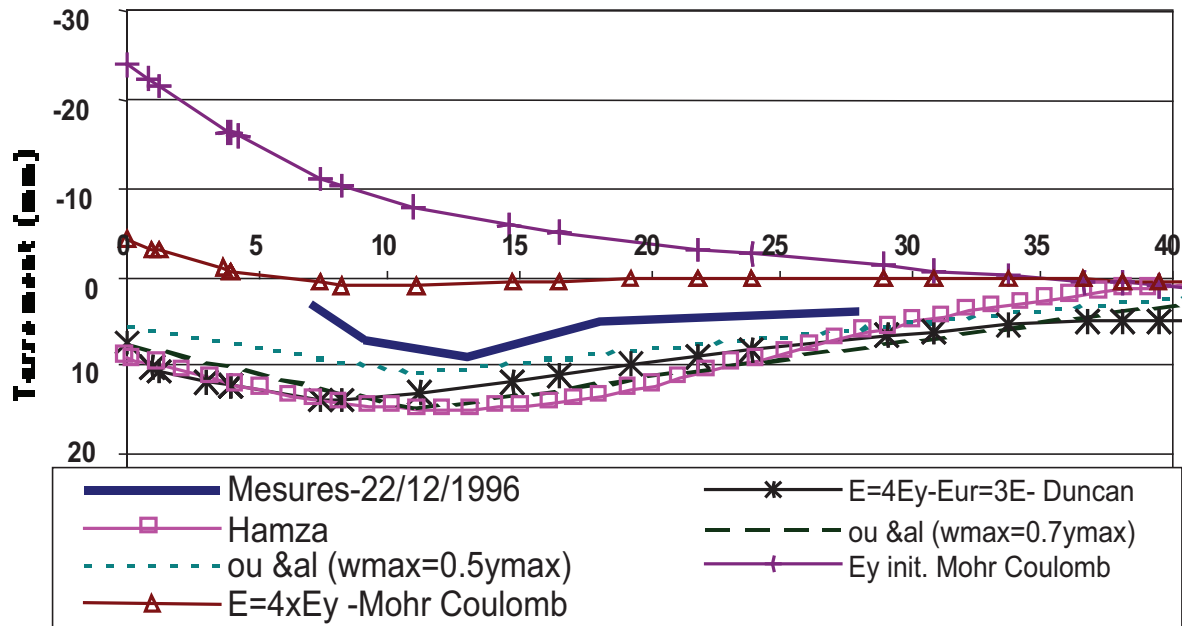


Fig. 12. Comparison between measured and calculated settlements using empirical, linear and non linear models, after Bazin and Schmitt (2001).

Fig. 12. Comparaison entre tassements mesurés et calculés à partir de modèles empiriques, linéaires et non linéaires, d'après Bazin and Schmitt (2001).

conventional ones, since less design assumptions appear necessary, whilst more information is generally provided by such models in terms of overall displacements, settlements, interaction between underground structures, and so on.

Nevertheless, as far as ULS design is concerned, FEM is generally used in conjunction with MFA rather than RFA, so that the same limitations apply.

With respect to SLS, main limitations come from the importance of soil investigations that are never sufficient, despite the necessity to rely on a sufficiently important series of laboratory tests, in order to provide all the parameters that may be required by the calculation model.

Meanwhile, there is a general consensus to facilitate the task by neglecting soil anisotropy, although anisotropy is known to be significant within sedimental layers: this may explain why it is not so easy to obtain representative results in terms of both horizontal displacements and settlements, as shown by geotechnical monitoring of the Grand Paris metro project (Fig. 10, from Daktera *et al.*, 2020).

This was also identified as the only explanation why it was never possible to match inclinometer data during both kinds of execution stages (see Fig. 11b), *i.e.*, excavation stages and prestressing of anchors, for the Colombes station project (Serrai, 2001; Plumelle *et al.*, 2005; Schmitt, 2009).

Indeed, stress paths involved in such projects are very specific, involving uniaxial unloading-reloading process behind the wall (*i.e.*, decompression during the first stages

when the wall acts as a cantilever, then recompression when anchors are prestressed), and biaxial unloading-reloading process under excavation (*i.e.*, vertical decompression due to excavation, and horizontal recompression when mobilizing passive earth pressure).

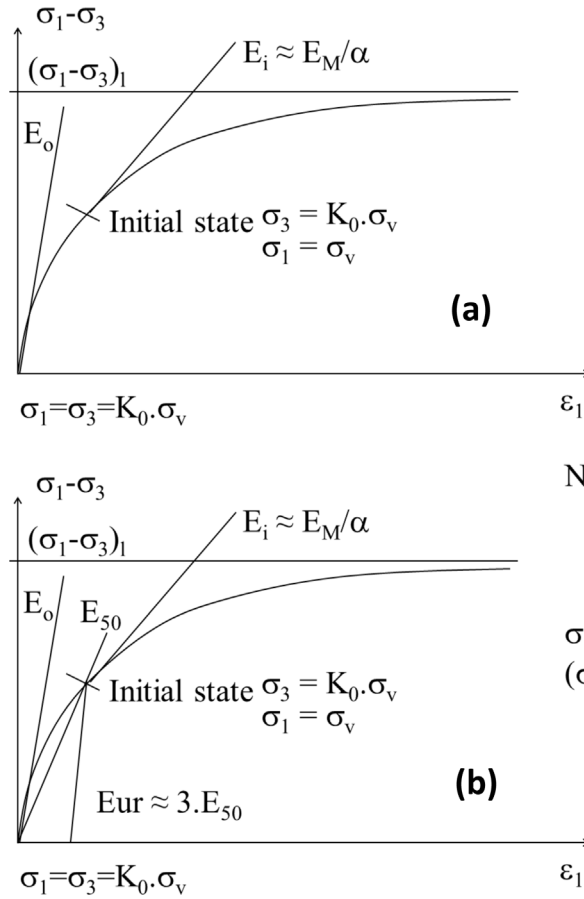
FEM models automatically account for these differently orientated stress paths on both sides of the wall, but do not consider the fact that their effects may partly be balanced by intrinsic anisotropy in soil properties themselves, that might explain why different sets of soil parameters were finally necessary to simulate the different stages.

This paradoxically was not necessary with subgrade reaction calculations (see Fig. 11a) that only model horizontal behaviour without paying attention to anisotropy.

5.2 Limitations specific to linear models

Many specialists consider that simplified models, typically those based on linear elasticity, should be avoided for the design of retaining structures, because of their inaptitude to properly consider counterbalanced effects of heave due to excavation inside and settlements earthside, by exaggerating effects of the former, as shown in Figure 12 (Bazin and Schmitt, 2001; Nguyen, 2003).

Indeed, this may no longer be justified if the purpose is not to calculate settlements, but only horizontal displacements and deflections of the wall, as per subgrade reaction models.



Normally consolidated soil :

$$\begin{aligned}
 K_0 &= 1 - \sin\Phi \\
 (\sigma_1 - \sigma_3)_0 &= \sin\Phi \cdot \sigma_v \\
 \sigma_{11} &= \sigma_3 \cdot (1 + \sin\Phi) / (1 - \sin\Phi) \\
 (\sigma_1 - \sigma_3)_1 &= \sigma_3 \cdot 2 \cdot \sin\Phi / (1 - \sin\Phi) \\
 (\sigma_1 - \sigma_3)_1 &= 2 \cdot \sin\Phi \cdot \sigma_v \\
 (\sigma_1 - \sigma_3)_0 &= (\sigma_1 - \sigma_3)_1 / 2
 \end{aligned}$$

Fig. 13. Graph of hyperbolic behaviour and soil moduli, after Philip *et al.* (2021).
Fig. 13. Loi hyperbolique et modules de référence, d'après Philip *et al.* (2021).

In such conditions, it may be shown that both models often provide very similar results (Ducas, 2001; Escobar, 2001; Azoune, 2002; Philip *et al.*, 2021; Millotte and Fonty, 2022), the former being possibly more reliable in specific cases where rigid passive struts are used, as described in Section 4.1, and the latter being possibly more reliable when prestressed supports as used, as discussed in Section 5.1.

The similarity of results provided by such different models, and their validity compared with geotechnical monitoring results, that may be considered as surprising, is particularly enlightened when soil parameters, more especially soil moduli, are derived from *in situ* tests, such as pressuremeter, that are frequently used for both calculations.

This may be considered as an opportunity to initiate a better understanding and a common language for SLS practice using various models.

Such understanding necessitates a preliminary definition of what may be considered as a “representative” soil modulus, which may not be so easy to assess as supposedly general diagrams, such as in Figure 8, generally assume.

As a matter of fact, how could one single diagram provide universal conclusions that would apply to the various laboratory tests devices, *in situ* tests and geotechnical structures, for which initial states, from which deformations are measured, are clearly not the same, and more especially to foundation or retaining structures that involve completely different stress paths?

The case of pressuremeter tests is very specific, since it is the only one that provides an *in situ* measurement of a horizontal soil modulus, but in purely deviatoric and short term conditions, to which some categories of soil are more sensitive than others: this needs to be corrected by the “structural coefficient” α , in order to derive what may be considered as a “Young modulus” in standard conditions, that also include isotropic stresses, such as triaxial tests.

Figure 13a identifies, in a triaxial diagram, the so-called Young modulus that Ménard intended to derive from E_M/α (Ménard, 1960; Gambin, 2005; Philip *et al.*, 2021).

This “tangent Young modulus starting from *in situ* initial conditions” (E_i on the figure) should clearly be distinguished from the “initial modulus in laboratory conditions” (E_o on the figure): the latter considers zero deviatoric stresses, since the soil sample has conventionally been isotropically reconsolidated, whilst the former considers *in situ* deviatoric stresses, as in the test itself and the project as well, typically $(1 - K_0) \cdot \sigma_v$ when considering a normally consolidated material.

In the same conditions, the plastic behaviour may be described by Coulomb’s theory, that results in a deviatoric stress equal to $(K_p - 1) \cdot \sigma_v$, with $K_p = (1 + \sin\Phi)/(1 - \sin\Phi)$.

Considering Jaky’s formula $K_0 = 1 - \sin\Phi$ as an order of magnitude, one may roughly conclude that deviatoric stresses at rest are about half of deviatoric stresses at failure (Plumelle *et al.*, 2005).

Thus, as shown in Figure 13b, $E_i = E_M/\alpha$ can be roughly assimilated to $E_{50\text{tangent}}$, that is $E_{50\text{secant}}/2$ if the soil behaviour may be considered as hyperbolic, as considered by the Hardening Soil Model (HSM) implemented in PLAXIS.

Although this is a very basic calculation, only intended at providing orders of magnitude, it may explain why back-analysis considering HSM are often successful provided that E_{50} values in the range of $2.E_M/\alpha$ are used (potentially higher values in over-consolidated soils).

We can also conclude that, if E_M/α may be considered as a representative modulus in the case of a monotonous loading, hence of a foundation project (there again in normally consolidated conditions, higher values being currently encountered in over-consolidated conditions (Baguelin *et al.*, 2009), as well as lower values in under-consolidated conditions (Lavisse and Schmitt, 2004; Combarieu, 2006), much higher representative values need to be considered for excavation projects.

As a matter of fact, such projects typically involve a significant unloading-reloading process, for which a representative modulus when a linear model is used should be roughly set between $E_{50\text{secant}} = 2.E_M/\alpha$ and $E_{ur} = 3.E_{50\text{secant}}$ (conventional order of magnitude) $= 6.E_M/\alpha$.

This may explain why back-analysis using a linear model is often successful provided that representative soil moduli in the range of $4.E_M/\alpha$ are used (potentially higher values in consolidated soils as previously recalled) (Phienweij *et al.*, 1998; Escobar, 2001; Serrai, 2001; Azoune, 2002; Plumelle *et al.*, 2005; Philip *et al.*, 2021).

This is also in line with successful predictions generally provided by subgrade reaction models using Schmitt's formula (1) that implicitly includes a representative soil modulus equal to $3.6.E_M/\alpha$ (see Sect. 4.2.1) that is the same order of magnitude.

We can now understand the reason why this empirical value of k coefficient is significantly higher than conventional ones: this is not at all the consequence of non-linearity, as initially assumed in line with Figure 8, but of the specific unloading-reloading stress path that characterizes excavation projects.

This may also explain why linear elasticity provides so realistic results when applied to the specific case of retaining structures, using either conventional or numerical models.

6 Conclusions

Finally, all calculation models and design approaches have limits, which are important to be aware of in order to meet both safety and cost efficiency requirements, based on results of geotechnical monitoring and practical experience.

Some results may be unexpected, such as the large extent of the validity area of simplified models, *i.e.*, linear elasticity and subgrade reaction coefficients.

But even if more remote than expected, it is of the utmost importance to know about these limits, and be able to proceed to necessary adjustments whenever project conditions or specific design situations make it necessary.

This article tried to review limits that have been detected so far, based on French experience and large databases of monitoring results.

It also tried to outline progresses that still need to be done in order to discern some limits that are still unprecise.

It outlined as well that there should be no debate in order to impose a ULS approach all over Europe, since both MFA and RFA are clearly relevant.

But it should be emphasized that each of them is especially relevant in specific project conditions and design situations that are not necessarily the same, so that both approaches are needed.

Indeed, knowing better the limits of approaches and models enables to pragmatically combine them rather than exclude some of them, finally resulting in safe and efficient designs, as well as a better understanding of practices in use in different countries.

It is now obvious that all practices cannot be unified, for the simple reason that they do not apply to the same conditions.

Nevertheless, we can reasonably expect that future releases of Eurocode 7 will be a significant step, probably not towards a universal approach of geotechnical practice, but rather towards a universal understanding of geotechnical practices.

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