NUMERICAL ANALYSIS OF EMBEDDED WALLS FOR DESIGN WITH EUROCODE 7

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1.- Introduction

1.1.-Embedded walls

Probably one of the most common features in civil engineering designs is that they interact with the ground. One common situation is that at some stage of the construction procedure lateral forces exerted by vertical or very tilted surfaces have to be resisted to permit a convenient development. That is the role of earth retaining structures.

The following three types of retaining structures can be distinguished: (EN1997-1:2004 9.1.2 (I))

- Gravity walls, in which the weight of the wall, sometimes including stabilizing masses of ground (stem walls), plays a significant role in the support of the retained material.

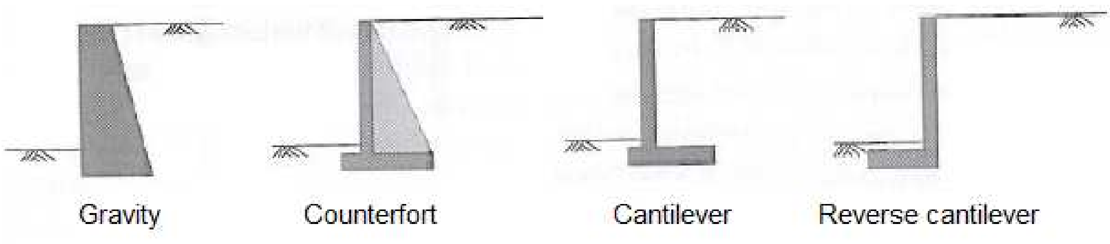


Figure 1.1. Gravity retaining walls. Reelaborated from Potts & Zdravkovic (2001). Chapter 3 75-76.

- Embedded walls, which are relatively thin walls of steel, reinforced concrete, or timber. These walls either rely for stability solely on the earth resistance due to the passive earth pressure in front of the walls (cantilever walls) or are supported by anchorages and struts (supported walls). The bending resistance of an embedded wall plays a significant role in the support of the retained material compared with the weight of the wall.

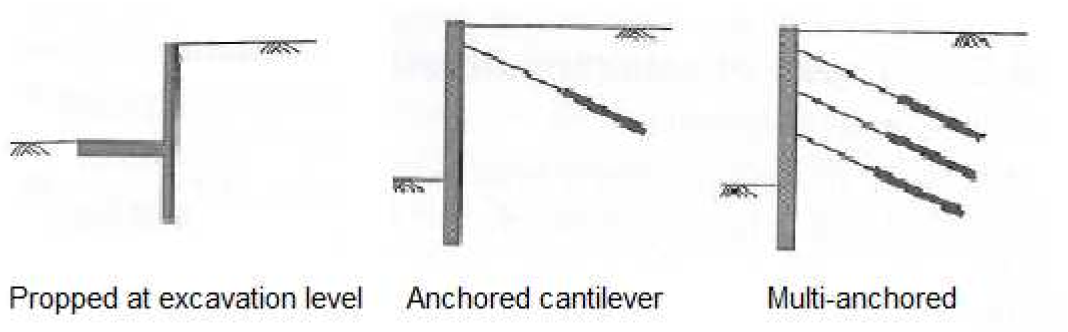


Figure 1.2. Embedded walls. Reelaborated from Potts & Zdravkovic (2001). Chapter 3 75-76.

- Composite retaining structures, which include walls combining elements of the previous two types. Typical examples are cofferdams and reinforced earth and nailed structures.

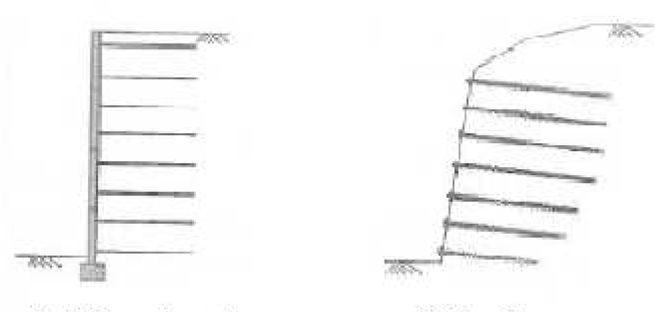


Figure 1.3. Composite retaining walls. Reelaborated from Potts & Zdravkovic (2001). Chapter 3 75-76.

Within these, the embedded walls are used to support the sides of deep excavations, quay walls, abutments, to limit ground movements and to control groundwater. This kind of earth retaining structure is widely used because of its high performance regarding the height achieved, respecting imposed space limitations, achieving small deflection tolerances and providing structural capacity. Sheet pile walls, diaphragm walls, contiguous bored pile walls and secant pile walls are examples of this type of wall (Potts & Zdravkovic, 1999)

1.2- Regulations in design of embedded walls

Eurocode 7 has been the primary geotechnical design code in Europe since 2010 (Potts & Zdravkovic, 2011). The aim of Eurocode is "the elimination of technical obstacles to trade and the harmonisation of technical specifications" (EN 1997-1:2004:P5). Ten interlinked standards provide structural design rules with the added value of making consistent the criteria for design in sub-structure and super-structure (Appendix A). Geotechnical design is treated in Eurocode 7 (EC7). This document consists of two parts: Part 1 explains the basis for geotechnical design and Part 2 defines field and laboratory testing.

All design should be satisfying the prescriptions for all design situations. These should be selected so as to encompass all conditions which are reasonably foreseeable as likely to occur during the construction and use of the structure (R Frank et al, 2004). The different design situations for ultimate and serviceability limit states are defined in EC7. Particularly in geotechnical design, assessment of mass permeability of saturated ground and its flow conditions compared to construction and use time might lead to both drained and undrained situations having to be considered for the design situations.

The durability is a relevant consideration in Eurocodes, understood as the ability of the structure to keep in an appropriate state for use in its design life, considering maintenance. Maintenance or even inspection is often difficult or impossible for embedded walls. Hence, aggressiveness of the environment and degradation of materials over time should be taken into account in design.

Controlling the level of risk in design leads to be able to make the appropriate decisions to balance the economic motivation of designs which need fewer resources (material, workmanship, time). This is covered by the codes of practise, like EC7, which regulate the engineering practise. Numerical methods are a powerful tool to be able to design for more complicated solutions and more optimised designs. Then, the treatment of the design values should be clear enough in order to satisfy the imposed regulation with the wished level of precision and accuracy, which the methods allow to reach. Part of this is the guidance missing in geotechnical design and some of its aspects are the points to be investigated for embedded walls.

The guidance to generate design values is not just a matter of defining the use of partial factors or just defining their values but involve reliable material characteristic values which are selected according to testing, structure type or experience; so it is a more general problem (Appendix A).

2.- Methods of Analysis of embedded walls

The design of an embedded wall will result in the determination of the embedment depth and the detailing of structural design (thickness, steel disposition in concrete). Furthermore, overall stability should be maintained, as well as the Ultimate limit states and serviceability limit states which apply and are described by EC7 (See Serviceability/Ultimate limit state design of embedded walls).

The traditional method for design is based in limit equilibrium method; however the use of numerical methods is becoming more common. The finite element method is generally accepted a tool for assessing the serviceability limit state for geotechnical structures whereas the factor of safety at the ultimate limit state is more commonly determined by conventional limit equilibrium methods. However, the different ways of implementing FEM result in discussion about the proceedings. FEM are the only tool which satisfactory evaluates displacements, and in general, the requirements of Serviceability Limit States. Its incorporation in the design practise raises the possibility to extend its application to Ultimate Limit State. Numerical analysis can deal with the requirements but is lacking in the definition of procedures for design practise. Schweiger (2005) states that numerical methods are indeed capable of evaluating the factor of safety for various types of problems but it is not clear if results from the applications of the finite element method to determine the factor of safety comply what is intended in respective standards and codes.

2.1.- Design of embedded cantilever walls with limiting earth pressure method

In general, in geotechnics, the complexity of the governing equations of the problem make it impossible to deal with the exact solution for traditional methods; therefore the need of simplifications when calculating embedded walls by means of hand calculations.

In this context, the limiting earth pressure method, as an application of limit equilibrium, has been widely used. The key consideration of this method is that the wall will rotate about a point near its toe and mobilise active earth pressure and passive resistance in the sides of the wall. In addition there is the possibility of accounting for pore water pressures, adhesion and friction or even external surcharges or anchors and props.

A series of assumptions are applied, and horizontal equilibrium and moment equilibrium about the toe are imposed. As a result an embedment depth is determined. To ensure the design is conservative an empirical correlation factor (~20%) increases the embedment depth. This allows for the geometric simplifications and analytical assumptions.

The determination of the bending moment distribution is carried out integrating the distributed forces along the depth of the wall as an elastic beam and imposing assumed boundary conditions. This assumption depends on previous knowledge of the behaviour of the wall from the ratio retained height – embedded depth. The two typical cases checked are known as free/fixed support. For the first one it is usually assumed zero reaction and moment at both ends of the domain. However for the fixed case the conditions are the top of the wall are the same but at the bottom the no reaction condition is replaced by the no horizontal displacement condition (rotation permitted).

The referred common assumptions of the method are:

- Rigid wall
- Soil rigid perfectly plastic
- Limiting lateral earth pressures apply
- Assumed failure mechanism: horizontal translation or rotation about a point

The method and the exact solution of the problem both require imposition of equilibrium, but the method omits compatibility conditions, boundary conditions and excessively simplifies the constitutive models such that the modelling of the soil-structure interaction is compatible. As a result, a poor modelling of the soil-structure interaction is obtained.

As a result of these assumptions one should consider the limitations of the method. It is known that the assumptions don’t occur in the whole domain, but are required to permit a hand calculation. This already highlights the shortcomings of the method and the potential advantages of a numerical approximation to the exact solution of the problem in terms of the mechanics of continua.

2.2.- Geotechnical design with numerical methods:

The design of structures with EC7 requires satisfying the conditions imposed by the Ultimate Limit States and Serviceability Limit States. The traditional tool used to solve the problem is the limit equilibrium method, which is based in strong assumptions, and consequently presents limited range of applications.

Alternatively, the advantages of finite elements methods are (Potts and Zdravkovic, 1999):

- Equilibrium and compatibility requirements satisfied (determination of deflections),
- utilisation of more advanced constitutive soil models,
- realistic in situ boundary conditions applied,
- definition of construction and working states in a single model,
- interaction between structures can be simulated,
- the analysis predicts behavioural mechanism. Schweiger (2005) highlights the potential of numerical methods in taking into account soil-structure interaction when investigating failure mechanisms. A significant effort of not having to check for all possible mechanisms is being saved to the designer.

FEM ideally suit calculations for serviceability limit state checks, where marked improved accuracy for predicting stresses, forces and deformations under working loads in soil-structure interaction problems is reached. However, the application for ULS which results depend on how partial safety factors are applied, is not clearly comparable to the definitions in codes.

Hence, it can be stated that finite element methods provide all the information required from the analysis, take into account all the principles governing the mechanics of the problem and allow to adjust in the design accurately and precisely the key parameters. To implement them for ULS, guidance on the applications of factors of safety from the codes is desirable.

2.2.1.- Finite element modelling for earth retaining structures

Numerical analyses are widely used in practical geotechnical engineering to assess the deformation behaviour of deep excavations, in particular when the influence on existing infrastructure such as buildings or adjacent tunnels has to be evaluated. It becomes increasingly common to use results from numerical analysis as basis for the design (Schweiger 2009).

A good modelling of the problem involves all relevant information of the soil and the structure taken into account; then the representative behaviour can be simulated. In the context of retaining structures, the following points deserve consideration:

- Symmetry. Real problems are 3D, the assumptions for a 2D representation (usually axisymmetry / plane strain) must not only be fulfilled in geometry but in stress distribution, soil properties, ground conditions and construction sequence.

- Geometry of the domain. The imposed boundary conditions are not exactly what reality present so the points relevant for the results should not be strongly influenced by the boundary conditions but better governed by the equations representative of the problem. This affects the choice of depth and lateral extent for the mesh. Symmetry might fix the position of some boundary. Stratigraphy gives guidance where to allocate the bottom, strong stiff layers (mechanical properties of which usually increase with depth) make the analysis less sensitive to the allocation of the bottom boundary. Different constitutive models present different accuracy depending upon the allocation of the vertical boundary; those which describe small strain effects accurately perform better.

- Support systems are not easy to introduce in the modelling. It should be distinguished the way in which are attached to the structure and the soil, and the way in which affect the system. Connections should respect compatibility/boundary (no displacement) conditions, active, passive and stress dependent systems should be taken into account differently. Some of them even show dependency with flow regimes. Details can highly influence bending moment distributions and displacements.

- Choice of constitutive models for structural materials and for the ground.

  Structural elements are usually represented with linear elastic models (enough for the range of stress in which are working). Unreinforced concrete behaves differently in tensile areas. Schweiger (2005) warns that one could be misled by a calculation of a high factor of safety when structural elements are treated as elastic material, an assumption which is often acceptable for investigating working load conditions. However for deep excavations this issue does not arise because diaphragm walls or sheet pile walls are not designed for plastic hinges, although this would be acceptable under the prescriptions of the code (Schweiger 2010).

  In the soil it is convenient to distinguish between backfill material, which is well represented by Mohr-Coulomb or Lade models; and in situ material from the excavation, for which models taking into account nonlinearity at small strains and soil plasticity show more accurate results. Schweiger (2010) shows that the choice of the constitutive model for the soil has a direct consequence for the design because different constitutive models lead to different design forces; and states that the simple elastic perfectly plastic models are not capable of representing the stress strain behaviour of soils correctly and therefore it remains questionable whether they should be used for design purposes.

- The construction method for excavation affects several variables of the problem: Initial soil stress is highly influencing nonlinear constitutive models, and construction may not respect the greenfield conditions. Execution of the wall modifies ground water conditions and introduces time related behaviour (consolidation coupled, change of stress conditions due to hydraulic gradients). In the case of needing dewatering conditions on the water supply, this will affect the flow regimes and consequently the analysis of the problem.

2.2.2.- Finite element modeling of embedded walls

Particularly, when applying FEM to embedded walls, it is important to consider:

Installation effects can be important to embedded walls, but most of the analyses assume that the wall is “wished in place”. If the construction holes are supported to minimise the displacements and hence the stress change is minimum, the former consideration is valid for modelling. Changes in stress are very localised with proper executing techniques. Calculations don’t usually take into account 3D redistribution. 2D plane strain analyses are likely to overestimate the effects of installation. Field measurements for this problem are not reliable, partly because there is little field information available.

The type of elements to model the wall can be solid elements –appropriate for thick walls > 0.8m – or beam/shell elements for steel sheet piles. Solid elements permit taking into account the moment generated in the wall by the shear stress transmitted from the ground on the back of the wall. The behaviour of the interface can be modelled with interface elements around the wall. In this elements the stiffness does not influence the results but the angle of dilation with which are modelled does.

Modelling the wall also includes assigning stiffness parameters. A long term reduction in stiffness for concrete walls as a consequence of creep and cracking successfully represents the increase in lateral displacements and the reduction in bending moments to withstand. The wall has to be assigned a value of permeability when dealing with groundwater (flowing). It can be treated as impermeable, permeable or can be given a value of permeability. The important point is to reflect the relative permeability of the wall in relation to the surrounding ground.

Support systems can be used in the design of embedded walls, and their modelling is uncertain in the following aspects: the effective stiffness, in which effort should be put for not overestimating, is dependent on temperature, creep, bedding, concrete shrinkage and deformable contacts. The time between excavation and installation of the support systems and the level of prestress applied in active supports are variables not easy to control. Those kind of supports require to monitor their movements and regulate the load applied.

The connection details of the support systems are very important because they determine if bending or tension can be transmitted and hence govern the behaviour of the system.

Special attention deserves the modelling of ground anchors, beam and interface elements should take into account the shear stress in the fixed length and a beam elastic tendon without frictional component should be placed for the free length. For the case of berms is better not to use the surcharge load but modelling them. For relieving slab is very important to represent realistically the complicated soil behaviour in the interface (only possible to proceed with a full numerical approach).

Long term behaviour is important to be taken into account for over consolidated clays even if it takes years to be the governing phenomenon. Post-construction effects can be reduced if drainage occurs during construction, especially if the process is slow. In case of being draining the soil, special attention should be put when stop draining it. For assessing swelling small strain constitutive models should be considered.

Schweiger (2010) shows that the numerical models of excavations emphasize the fact that elastic-perfectly plastic constitutive models such as the Mohr –Coulomb model are not well suited for analysing this type of problems and more advanced models are required to obtain realistic results. Reasonable lateral wall movements may be produced with simple failure criteria with appropriate choice of parameters. However vertical movements behind the wall are in general not well predicted, obtaining heave in many cases instead of settlements. Strain hardening plasticity models including small strain stiffness behaviour produce settlement thanks to being more in agreement with the expected behaviour.

3.- Design with Eurocode 7

Design, execution and in general all the stages of construction are regulated by codes of practise. The regulations for embedded walls are compiled within Eurocode 7 (EC7- EN 1997); particularly Section 9 treats retaining structures. This document is the code of practise which should be complied. EC7 includes mandatory rules and application rules stating the basis for design and the requirement for testing, all these with the aim of reaching the harmonisation of the design practise. However, in order to respect differences for the allowance in the designs in different places with different experiences in construction; parameters such as partial safety factors are specified on the National annexes.

Instead of the standard lumped factored approach to geotechnical design Eurocode establishes limit state design:

Traditional geotechnical design practice uses a lumped global factor of safety to prevent failure and/or ensure that the settlements under working loads are within tolerable levels (Discroll et al, 2008 as cited by Blackwell, 2010).

In contrast, the limit states are defined by a performance criteria which is expressed in terms of the fundamental inequality:

Design value of efect of action < Design value of resistance (OR tolerance defined)

To fully understand and be able to apply the expression above, the following concepts apply:

- Actions: Imposed boundary conditions to the structure. Specified geotechnical actions are imposed on a structure by the ground, fill or water. They include the soil's self weight, earth pressures and ground water pressures, surcharges, seepage forces, traffic loads, structural loads, swelling, temperature effects. (EN1997-1:2004 2.4.2(4))
- Structural response to actions are effects of actions.
- Resistance is the capacity of a part or the whole of a structure to withstand actions without mechanical failure occurring (Blackwell, 2010). This incorporates the ground capacity to resist actions as well as bending, buckling and tensile resistance within structural members. (EN1997-1:2004 1.5.2.7).

- Material characteristic values selected are a cautious estimate of the value associated with the limit state (EN1997-1:2004 2.4.5.2). The characteristic strength is the best estimate of the soil's strength from the available site investigation data (Potts & Zdravkovic, 2011). Annex A explains briefly how to obtain characteristic values.

The stated inequality must be satisfied for every limit state, for all action combinations, in all the situations identified during lifetime. The situations to consider, the definition of concepts like action, effect of action, and resistance; how to calculate them, and how to obtain the data to calculate are the guidelines provided by the code of practise. The data for the calculations are characteristic values of parameters and the results to be compared are design values. It should be noted that at some stage of the calculation characteristic values are transformed into design values by the application of a partial safety factor, which takes into account uncertainties and inaccuracies in the parameters and geometrical variability to ensure a safe design. The design approaches are the rules to guide that step.

The application of partial factor in strength parameters implies:

$$C_{u,design} = \frac{C_{u,characteristic}}{F.O.S.}$$

$$\tan\left(\emptyset'_{design}\right) = \frac{tan(\emptyset'_{characteristic})}{F.O.S.} \qquad c'_{,design} = \frac{c'_{characteristic}}{F.O.S.}$$

where C_u is the undrained shear strength, ϕ' the friction angle, c' cohesion parameter and F.O.S. the partial factor of safety involved.

3.1.- Partial factoring

The benefits and drawbacks of the limit state design with partial factors (Blackwell, 2010):

+ Separation of the ULS and SLS design cases, i.e. uncoupled safety and deformation analysis with clearly defined performance criteria.

+ Compatibility with structural design codes – coupled structure and soil design.

+ Local knowledge is retained as each member state determines its national parameters.

+ Individual risks and uncertainties can be applied to each component, and combined to give the worst effect.

+ Favourable and unfavourable actions are factored separately. E.g. opposing forces can’t completely cancel each other out.

- Multiple partial factors lead to confusion and additional calculations.
- A defined partial factor can be applied to a range of values e.g. unfavourable actions, where sources have different levels of reliability but the factoring implies the same value of assumed confidence.
- Failure to clearly define the characteristic value of peak, critical or residual strength undermines the subsequent application of a partial factor.

3.2.- Design approaches with Eurocode 7

Design approaches (DA) define how and which partial safety factors are distributed between actions, ground properties and resistances. It also outlines the ways to apply them into the fundamental inequality.

Schweiger (2005) states that the various design approaches differ in the way the partial factors of safety are applied to soil strength, resistance and different types of loads (actions). Although the original aim of EC7 was to ensure a unified design approach throughout Europe; it is unfortunately left to the national annexes which of the suggested approaches will be relevant for a particular country. In this context, although numerical methods are mentioned in EC7 as a possible design tool, not all of the design approaches can be applied in a straightforward manner within the framework of numerical modelling.

DA1 separately looks at failure in the soil and failure in the structure using two combinations of sets of partial factors. The partial factors are applied at an early stage to the representative values of the actions and soil strength parameters except if this leads to physically impossible situations, case in which are applied to the effect of the action; or for piles and anchorages, cases in which are applied to the calculated resistances. Combination 1 usually conditions structural sizing and focuses on design against unfavourable variability of actions, while design values of ground properties are equal to their characteristic values. Combination 2 governs geotechnical sizing, factoring less restrictively actions but also factoring ground strength parameters and resistances. This is the recommended approach in the UK.

In DA2 a set of partial factors is applied. It can be applied to the ground resistance and the actions or to the ground resistance and the effects of the actions.

In this last case $\gamma(E)E < R/\gamma(R)$ so the overall factor of safety is $\gamma(E)\cdot\gamma(R)$. In the implementation of this procedure various components of the actions (i.e. permanent, variable) account for different partial safety factors and the separation is not transparent. This is commonly referred as DA2*

One way of dealing with DA2 could be that the analysis is performed in terms of the unfactored strength parameters for the soil; and the correspondent bending moments, anchor forces and passive resistance is factored by the respective partial factor of safety in order to produce design values. However, due to nonlinear soil behaviour this is not quite what DA2 intended to be, although differences are probably not very significant for internal forces may be significant in terms of passive resistance (Schweiger 2005).

When calculating design values of effects of action using DA2* a linear approximation is performed. Let M1 and M2 the values of the variable e.g. bending moment with and without the variable load applied (generally M1 > M2). Then:

$$M_{design} = 1.35 \times M2 + 1.5\ (M1 - M2)$$

DA3 factors characteristic values of actions from the structure separately to the geotechnical actions, which are factored by the strength parameters. Also proposes factoring soil strength parameters in terms of resistance.

Design Approach	Actions		Soil strength parameters			Passive resistance
	Permanent unfavourable	Variable	tan(ϕ’)	c’	C_u	
DA 1/1	1.35	1.5	1.00	1.00	1.00	1.00
DA1/2	1.00	1.30	1.25	1.25	1.40	1.00
DA2	1.35	1.50	1.00	1.00	1.00	1.40
DA3	1.00	1.30	1.25	1.25	1.40	1.00

Table 3.1 . Partial safety factors recommended by EC7 for each design approach.

3.2.1.- Differences and issues with design approaches

The main difference between the methods is whether the partial safety factors are applied to a “primary” variable, e.g. the material properties and applied actions (DA1 and DA3), or to the “secondary” results i.e. calculated values including the resistances and action effects (DA2). (Discroll & Simpson, 2001 as cited by Blackwell, 2010). Schweiger (2005) analyses an embedded propped wall and concludes that when investigating the different approaches, results are reasonably consistent with respect to design bending moments and strut forces but design values for passive resistance are more difficult to assess when different approaches are compared.

Given the uncertainties inherent in any analysis in geotechnical engineering the differences due to the different approaches seem acceptable provided a suitable constitutive model is employed (Schweiger 2010).

The reason for applying the factors of safety as close to the uncertainty as possible is the nonlinear relationship between action and effect of action, making sure the design values are a safe estimate. This nonlinearity is significant in embedded retaining wall design, earth pressures depend nonlinearly on tan (ϕ), and in addition, bending moments increase nonlinearly with earth pressures. Consequently it would be preferred to factor tan (ϕ) rather than the bending moments.

On the other hand, if applying factors at the source is not realistic (in pore water pressures) and result in physically impossible stress states then factors are applied not to the action but to action effects.

It should be noted that the different procedures are inconsistent when it comes to passive earth pressures. These are factored as earth resistances in DA2 (factor 1.4) but left unfactored (factor 1) in DA1 and DA3 as are considered favourable geotechnical actions. Values for active and passive earth pressures based on DA2 do not correspond to the results from DA3 analysis. The differences in earth pressure distributions can be interpreted justifying that due to the reduction in soil strength the passive pressure increases slightly in the analysis due to a higher mobilisation caused by increasing wall deflection (Schweiger 2005). It should be noted that this issue does not appear in limit equilibrium because the factoring of the passive earth pressure is possible separately as is treated as an input of the equilibrium method rather than an intermediate result.

When analysing the application of the design approaches in numerical modelling, Schweiger (2010) points out that DA1 is basically a combination of DA2 and DA3. This means that the application of a partial safety factor associated with a particular action, soil strength parameter or resistance is performed either in the same manner as DA2* or DA3. Consequently, it could be it could be considerate a conceptually intermediate approach. However, one must consider that the results are not linear combinations, so differences like the effects of the water pressure being fully factored in DA2 whereas they are not in DA3 can lead to important differences between the approaches (e.g. pore water pressures at both sides of the wall in some cases will cancel each other or they will not if factoring on them is applied.)

3.3.- Accounting for partial factors of safety in numerical analyses

The design approaches can be implemented by means of two methods when considering finite element analysis (Bauduin et al, 2000):

1. Load and Resistance Factoring Approach (LRFA): Actions, action effects and characteristic values of resistances are factored.

This approach is more convenient when considering that the full load history can be simulated. Factors are applied for the checks but all the history dependent variables are simulated with its characteristic values.

One problem when using numerical methods arises because DA1/1 and DA2 require permanent unfavourable actions to be factored by a partial factor of safety, e.g. the earth pressures acting on structural elements such as retaining walls and tunnel linings. This is not possible because in numerical analyses the earth pressure is a result of the analysis and not an input (Schweiger 2010).

Eurocode 7 allows for applying the partial factor on the effect of the action instead on the action itself. This is commonly referred as DA2*. Numerical methods can be applied because analysis is performed with characteristic loads and characteristic parameters introducing the relevant partial factors at the end of the analysis.

It can be concluded that in principle all design approaches specified in EC7 can be used in combination with numerical modelling provided that DA2 is used in form of DA2*. (Schweiger 2010).

However, Bauduin et al (2005) as cited by Blackwell (2010) stablishes that LRFA is restricted to structural members with linear behavioural. i.e. no plastic hinges, and is unable to deal with the complexities of models where the actions also affect the resistances e.g. passive pressures.

Given the great importance of the passive pressures in the analysis of embedded walls, especially those cantilevered, this method of analysis is discarded.

2. Material Factoring Approach (MFA): Ground strength parameters (ϕ', c' C_u) are factored as being the main source of uncertainty.

This approach gives structural forces which are directly obtained from the analysis and is actually applicable in most design situations.

This is the approach chosen for this investigation. For the calculations involved ϕ' is the only parameter to take into account, the analyses are assumed in non cohesive sand.

Authors like Bauduin et al (2005) propose that this method should incorporate a stepwise increase in the actions as well as decrease in the ground strength. This appears particularly onerous and contradictory to Design Approach 1 in EC7 (Blackwell, 2010).

3.3.1.- Implementation of Material factoring approach; AP1 and AP2.

For the material factoring approach two methods are available to factor soil strength in numerical analysis:

Increase of factor of safety or c'- ϕ' reduction, approach AP1.

Unfactored strength paramenters are used in each stage to perform the analysis, what actually represents SLS case. To check ULS the software undertakes a reduction in c'-ϕ'- C_u until equilibrium cannot be satisfied.

The use of this method implies using the same reduction factor for all the strength parameters, factor which is assumed to be the factor of safety of the stage. This can be contradictory with EC7, which specifies that different factors should be applied in certain cases e.g. analysis in total stresses with inter-bedded granular and cohesive material strata. In fact, where c' is zero or either for effective stress analysis this simplification is not significant.

Schweiger (2005) states that the safety factor resulting from a finite element analysis assuming a Mohr-Coulomb failure criterion can be obtained by reducing the strength parameters incrementally, starting from unfactored values until no equilibrium can be found in the calculations. Then the safety factor is defined as

$$F.O.S. = \frac{\tan\left(\varphi_{available}\right)}{\tan(\varphi_{failure})} = \frac{c_{available}}{c_{failure}}$$

In this investigation no other than actions from the ground are applied so additional factoring has been dismissed.

Initial factoring, approach AP2:

The inputs of the analysis are factored strength parameters. Hence, all values through each construction stage are considered ULS design values. All stages of the analysis may be completed without reaching failure, which ensures the stability of the problem, but it does not produce information on the real magnitude of the safety factor (Potts & Zdravkovic, 2011).

Issues involved with this method:

- SLS checks have to be performed separately.
- Not easy to use in combination with advanced constitutive models in which strength is stress and/or strain dependent (Potts & Zdravkovic, 2011).
- The initial conditions, which can be dependent on the strength parameters (e.g. assumed initial stresses by approximation of $K_0 = 1 - \sin\phi'$) are not modelled correctly. The disadvantage is that such a reduced strength may require inital stresses which are not consistent with those in situ (e.g. the earth pressure coefficient at rest , could be reduced). As a consequence, smaller structural forces could be calculated in, for example, retaining walls or tunnel linings that are present in the analysis (Potts & Zdravkovic, 2011).

An issue raised by the material factoring approach is that a decision with respect to initial stresses has to be made: If $K_0 = K_0\ (\phi)$ e.g. Jaky (1948) $K_0 = 1 - \mathrm{sen}(\phi)$; then it can be use the characteristic value for the friction angle or the design value. This is one of the points of the present investigations. It should be noted that for certain conditions K_0 based on the characteristic friction angle may violate the yield function (Schweiger 2010)

Method AP1 is preferred when construction sequence and load history are influential (Bauduin et al, 2000). Method AP2 is interpreted as a check for worse than anticipated parameters based on code defined factors. In contrast, method AP1 is providing the available factor on the mechanism.

3.3.2.- Principles of numerical application of the factor of safety in AP1

Due to the lack of guidance addressing this question in Eurocode 7, different software account for the c'- ϕ' reduction in different ways and mostly only for simple constitutive models (Potts & Zdravkovic, 2011). Moreover, the algorithms for different constitutive models might significantly differ. However, a methodology by Potts & Zdravkovic (2011) for accounting for partial factors in finite element analysis can be applied to any constitutive model; this is the implementation of ICFEP, the program used for the calculations in this project.

The accounting of the factor of safety in this method is implemented by:

A) Deriving a new relationship between the change in stresses and change in total strains.

B) Modifying the governing finite element equations.

The factor of safety in a constitutive model can be considered as an additional state parameter in the yield function $F\ (\{\sigma\},\{k\},F_s) = 0$ with a default value $F_s = 1$, to increase incrementally at a desired state of analysis.

With this introduction, the relationship between changes in stresses and total strains will account for the factor of safety (see Potts & Zdravkovic, 2011). It can be written as:
$\{\Delta\sigma\} = [D^{ep}] \cdot \{\Delta\varepsilon\} - \{\Delta\sigma_c\}$

Where the $[D^{ep}]$ matrix is identical to that of an elasto-plastic model which does not include a factor of safety and $\{\Delta\sigma_c\}$ represents the reduction in stresses due to the reduction of material strength (increase of F_s). For constant F_s, then $\{\Delta\sigma_c\} = 0$.

The governing finite element equations will account in the vector of forces for a third term in addition to surface and volumetric forces, term which is linear in $\{\Delta\sigma_c\}$ and which represents the effect of changing the factor of safety.

This formulation requires a derivative of the yield function with respect to the factor of safety which will be dependent on the constitutive model , but computable in any case.

As long as the dilation angle can be defined as a proportion of the friction angle, this relationship is kept during the increase of factor of safety in order to respect the constraint $v < \phi'$ (Mohr Coulomb constitutive model).

It should be noted that for advanced constitutive models the reduction in parameters is still referred to c', ϕ' (or C_u). This means that a relationship between the parameters of the model and c', ϕ' (or C_u) is needed for the implementation of this approach. With the method exposed, the relationship is implicitly introduced in how F_s relates to the parameters in the yield function.

3.4.- Serviceability limit state design of embedded walls

The typical limit states considered when designing are:

- Movement of the structure affecting its functionality or appearance.
- Modification of the flow regime of groundwater.
- Seepage.
- Cracking of the structure

The design in serviceability should not be assuming that the earth pressures get limiting values. Special emphasis should be put in the control of the ground displacements to account for this. The stiffness might be strongly related to the level of strain, the mode of deformation and the ground anisotropy. The lack of information about the stiffness might lead to be wishing to validate the design for both upper and lower estimates (in fact the characteristic values of the stiffness are the mean values, not taking into account its variability because depending in the case the unfavourable deviation can be positive or negative). As well the construction and the support installation sequences can influence significantly the initial stress conditions and are more important to be taken into account in the serviceability deflection check.

It should be noted that calculations in serviceability do not involve partial safety factors (different from one). Combinations of actions are different than in ULS. In SLS the design values are the same as the characteristic values and no conflict with the applications of safety factors appears when implementing numerical approaches. Actually, under working loads the assumptions of traditional methods on ULS are far from taking place, and this highlights the suitability of Finite Element Analysis for this checks. FEA have been widely accepted as a powerful tool to solve boundary value problems in geotechnical engineering. The serviceability limit states (SLS) i.e. prediction of deformations, stresses and internal forces under working load conditions, for complex interaction problems can be assessed only by numerical modelling (Schweiger, 2005).

3.5.- Ultimate limit state design of embedded walls

The typical limit states considered when designing are:

- Overall stability.
- Lack of equilibrium: Excessive rotation or horizontal translation or lack of vertical equilibrium. Might be not governed by material strength.
- Failure of structural element or connector between elements, including pull-out failure in the ground. Consider fatigue if time-dependant effects or actions are likely to appear.
- Hydraulic heave, internal erosion, piping, unacceptable leakage of water, or transport of soil solid particles. (Control hydraulic gradient)
- Uplift.
- Geotechnical failure or excessive deformation of the ground

The aim of the design is to find the minimum wall penetration in which no rotational failure occurs and vertical equilibrium is satisfied, and the structural design which withstands the distribution of effects of the actions and reactions.

Typical steps for design are:

i. Determination of geometrical characteristics, accounting for unforeseen overdig at excavation.
ii. Determination of pore water pressures and its resultant
iii. Geotechnical ULS analyses. Equilibrium, determination of wall-ground interface behaviour.
iv. Judgement of results, consideration of increasing the wall penetration depth up to the values of bending moments, reactions at supports, movements or hydraulic behaviour –if so repeat the first stages-
v. Structural design of the wall
vi. Consideration of other ULS like stability of anchorages, overall stability, hydraulic failure, vertical failure of the wall.

3.5.1.- Analysis of embedded walls for ULS checks

Soil – structure interaction

Designing embedded walls requires analysis of the interaction between the ground and the wall for determining earth pressures. The reason for this is that earth pressure can be stiffness controlled for small displacements or strength controlled once yielding is reached. Failure does not necessary occur in one or other of the situations, what is more, different points of the wall might be in different situations.

Depending on the toe penetration, embedded walls are designed for a free or fixed condition of earth support at the toe, which is governing the distribution of displacements along the height and hence the distribution of the earth pressures.

Different calculation models are used for design of embedded walls; can be grouped in:

a) Limit equilibrium methods or methods with assumed earth pressures. Only as long as the assumptions taken are fulfilled the analysis is valid, with the inconvenient that in many cases not taking into account compatibility conditions implies not having tools to check if the assumption is valid.
b) Methods which take into account wall-ground interaction, like beam models on ground nonlinear spring supports or finite element models. Finite element models treat the ground as continuum and respects compatibility conditions in the interface. To improve the pressure-displacement assumptions this models take into account constitutive equations of the soil defining stress-strain relationships. Also solving equilibrium and permitting to respect the boundary conditions ensure that the solution of the model is the solution of the problem in terms of mechanics of continua. Pore pressure can be also solved coupled within the equations of the problem. As a result no inconsistencies within the variables appear.

One key point for an appropriate soil-structure interaction modelling in Ultimate Limit State is the hypothesis upon the mobilised horizontal displacement; which leads to the determination of the earth pressure at that point assuming a certain stiffness or yield. For methods in a) if the hypothesis in displacement occurs under the real ultimate limit sate conditions then the result (safety, load) is acceptable. The fact that some traditional methods omit compatibility conditions make that the hypothesis cannot be checked within the calculation but is accepted based on assessment of the kind of mechanism and the knowledge and experience about it.

For the methods described in b) the displacements can be calculated and checked.

The earth pressure prediction in the spatial domain where displacements are not mobilising full strength at yield is usually complicated because the stiffness varies nonlinearly with displacement and over the depth.

EN1997-1:2004 provides guidance of the “at rest” and limiting earth pressure in Annex C. Earth pressures encompass the actions from the ground and groundwater. Their magnitudes are influenced by surcharges, wall inclinations, water levels, seepage, wall movement relative to the ground , strength of the ground and wall roughness (EN1997-1:2004 9.5.1(3)P)

Other considerations

Other points that apply to ULS of embedded walls should be noted:

- Another important consideration governing the soil and structure interaction is concerning wall friction and adhesion. The interface parameters are usually defined from ground strength and wall material properties; $\delta = k \cdot \phi$. The constant depends on the material of the retaining structure as well as in the construction procedure proposed, taking values between 0 and 1.
- ULS is related with the most unfavourable conditions that may occur within the life of the structure, so as for the determination of groundwater pressures. Design values can be derived either by applying partial factors to the characteristic water pressures or by applying a safety margin to the characteristic water level (EN1997-1:2004 2.4.6.1 (8)P).
- Surcharges shall be considered when they act on the retained surface (EN1997-1:2004 9.3.1.3 (1)P).
- The design situation should consider variation in soil properties, geometry and combinations of actions in space and time. (EN 1997-1:2004 9.3.3.1 (1)P).
- The design should take into account the possibility of over-dig up to 10% of the retained height between supports (or full retained height). Dismissing this allowance is specially unfavourable in the case of soils with high friction angle.

4.- Mohr –Coulomb constitutive model

The formulation of the constitutive model described below follows the approach from Potts & Zdravkovic, 1999 presented in Chapter 7 “Simple elasto-plastic constitutive models”:

The mohr-Coulomb model is an elastic-perfectly plastic model which is based in the linear failure criterion $\tau_f = c' + \sigma'_{nf}\tan\varphi'$ where τ_f and σ'_{nf} are the shear and normal effective stresses on the failure plane.

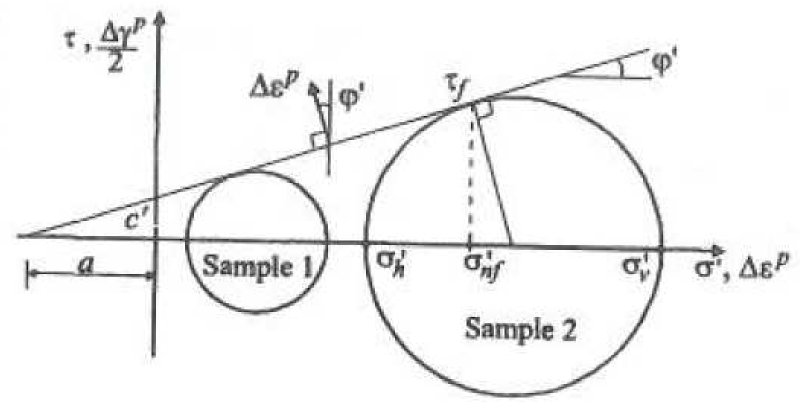


Figure 4.1 . Mohr circle’s of effective stress. Potts & Zdravkovic (1999) Figure 7.5 152.

Using the Mohr circle of stress and formulating in terms of the in-plane principal stresses the failure criterion can be adopted as the yield function:

$$F(\{\sigma\},\{k\}) = \sigma'_1 - \sigma'_3 - 2c'cos\varphi' - (\sigma'_1 + \sigma'_3)sin\varphi'$$

And in terms of stress invariants:

$$F(\{\sigma\},\{k\}) = J - \left(\frac{c'}{\tan\varphi'} + p'\right) g(\theta)$$

$$with\ g(\theta) = \frac{\sin\varphi'}{\cos\vartheta + \frac{sin\vartheta sin\varphi'}{\sqrt{3}}}$$

As the model is assumed to be perfectly plastic no hardening/softening law is required; the state parameter {k} is assumed constant and independent of plastic work or plastic strain.

An associated flow rule can be adopted. Then this results in tensile plastic strains, the strain increment vector is inclined at an angle ϕ' to the vertical. This means dilatants plastic volumetric strain. In this case the angle of dilation is equal to the angle of shearing resistance.

The angle of dilation can be defined: $v = sin^{-1}(-\frac{\Delta\varepsilon_1^p+\Delta\varepsilon_3^p}{\Delta\varepsilon_1^p-\Delta\varepsilon_3^p})$

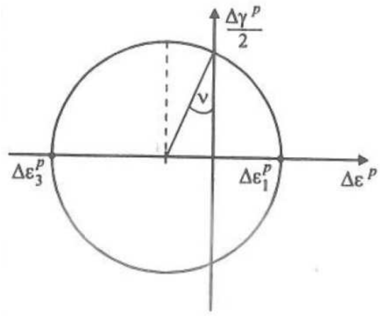


Figure 4.2 . Mohr's circle of plastic strains.
Potts & Zdravkovic (1999) Figure 7.7 153.

The issues with this approach are:

- The dilation is much larger than observed in real soils
- Once the soil yields it permanently dilates

A non-associated flow rule can rectify the first. This results in a fixed yield surface in the stress invariants space with a moving plastic potential surface through the current stress state. By prescribing the angle of dilation, the predicted plastic volumetric strains can be controlled. In this context v < ϕ'.

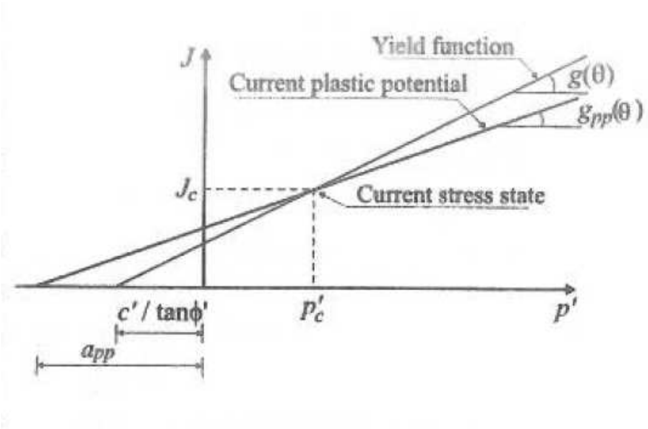


Figure 4.3. Relationship between the yield and plastic potential functions.
Potts & Zdravkovic (1999) Figure 7.8 154

To solve the second, the angle of dilation should be allowed to vary with plastic strain and a constant volume condition at large strains would be imposed (zero dilation).

However, this last approach is not implemented in the constitutive model used in this project.

The constitutive model incorporates as well E’ and μ’, parameters which control the elastic behaviour. In addition to c’, v and ϕ’ this adds up to 5 parameters to define the model.

Results clearly emphasize that elastic-perfectly plastic constitutive models such as Mohr-Coulomb are not well suited for analysing excavation problems and more advanced models are required to obtain realistic results (Schweiger 2010b). However, when using other constitutive models not defined in terms of c’ and ϕ’then application of the factor of safety has to be extended to the model parameters. This extension is not always transparent and for every constitutive model more than one option can be available. Again no guidance is provided in EC7.

Schweiger (2010) shows examples of analysis of deep excavatins in which the differences in results coming from different design approaches are more pronounced for the MC-model than for more advanced HSS-model (Hardening Small Strain). The reason for this behaviour is that a reduction is strength has a different effect in a linear elastic-perfectly plastic model than in a advanced hardening plasticity model due to the different stress paths followed.

Advanced constitutive models are capable to capture the stress-strain behaviour of soils for stress levels ranging from working load conditions up to failure with reasonable accuracy (Schweiger 2010). One important difference in the predictions is related with the deformation of the retained material. While Mohr-Coulomb models predict heave, the real behaviour and predictions of more advanced constitutive models present settlement.

Schweiger (2009) presents examples with which indicates that both design approaches DA2 and DA3 and consequently DA1 (the author treats this approach as a combination of the previous) can be applied in combination with the finite element method for ULS checks. In this examples the differences in result coming from the design approaches are in the same order (or smaller) than the differences in result due to the choice of constitutive model.

In this study the Mohr –Coulomb model has been selected, which allows an easier interpretation because the strength factoring does not need to be extended to other parameter models than those which govern failure. In addition, the model is particularly appropriate for the focus on the study of effects of dilation for different values of the friction angle; permits a more straight forward interpretation of results. The fact that no field data is available to compare with the numerical results reduces the value of precision in predictions against the conceptual matter of the investigation. The calculations developed aim to show result with which undertand and assess firstly the simple models. It should be reminded for further study that the methodology for accounting for partial factors in finite element analysis implemented in ICFEP can be applied to any constitutive model, and that more results with this models are required to validate this investigation.

4.1.- Effect of dilation in geotechnical calculations

It is also relevant to this project to illustrate the effect of dilation in calculations. The dilation controls the magnitude of the plastic volumetric expansion with shear.

In hand calculations the dilation is rarely taken into account, only in certain upper bound methods involving energy dissipation. The dilation is incorporated when calculating rates of work i.e. the direction of sliding forms the dilation angle with the plane where failure is assumed to occur so that the forces contributing should be projected parallel to the deformation. The consequence of this point is that admissible slip lines are straight lines or logarithmic spirals, for soil presenting constant dilation in the domain. (Prat, 2009)

Numerical methods have shown that dilation does not make a difference in volumetric unconstrained problems (e.g. Shallow foundation). However for volumetric constrained problems, computing assuming dilation leads to monotonic increasing load – displacement curves, which would imply that failure is not reached for any load level. This is the case of undrained analysis, where the total zero volume change constraint has to be met for the whole domain.

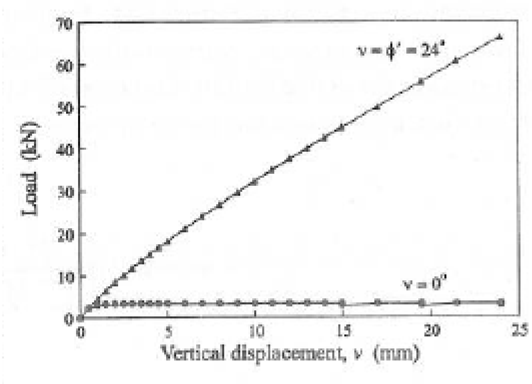


Figure 4.4 . Load-displacement curves for a strip footing, using the Mohr-Coulomb model with different angles of dilation. Potts & Zdravkovic 2001. 383.

It can be appreciated in the figure that for volumetric constrained problems in soils modelled with a Mohr – Coulomb model accounting for dilation also for large strains the failure load is not reached. A limit load is only obtained for zero dilation. Hence, it can be shown that the modelling of dilation can have an important role. Therefore it should be questioned if applying a partial safety factor on it is another missing guideline in Eurocode 7.

In this context it is important to bear in mind that actually, real soils reach critical state at constant volumetric strain, what suggests that the dilation has to be imposed zero at the failure of stress-strain. However, Mohr – Coulomb model assumes constant dilation, which can result in unrealistic predictions.

In the exposition of the constitutive model it has been stated that the angle of dilation is expected to be smaller or equal than the friction angle. When implementing the factoring of strength parameters this is the reason for also applying on it the safety factor. In Schweiger analyses (2010); it is stated that the angle of dilation is also reduced by the partial factor which is however not explicitly mentioned in EC7. It should be noted that when reducing the dilation angle because of applying AP1, the procedure is also being consistent with tending to impose a smaller dilation angle for the last stages before failure occurs; in the limit the soil would fail at zero volumetric change as in reality. This procedure is the implemented in the calculations.

5.- Details of the numerical simulation

This section aims to explain in detail the features of the analyses undertaken in this project; input parameters, assumptions and decisions made for simulating the retaining wall by means of ICFEP (Imperial College Finite Element Program).

5.1.- Fixed inputs

5.1.1.- Geometry

The basic geometry is presented in the figure below. The model accounts for symmetry about the centre-line of the excavation:

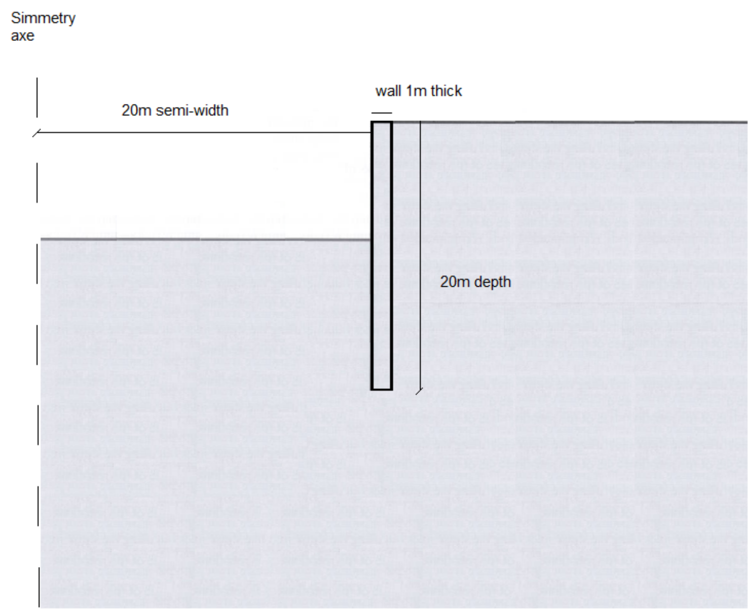


Figure 5.1 . Basic geometry of the problem (not to scale).

The presented geometry aims to model a broad excavation. Mechanisms of failure mobilised in this domain are expected not to interact with the symmetry boundary allowing for a more direct interpretation of the results of the model to the real phenomenon.

The geometry is introduced by means of the finite element mesh below.

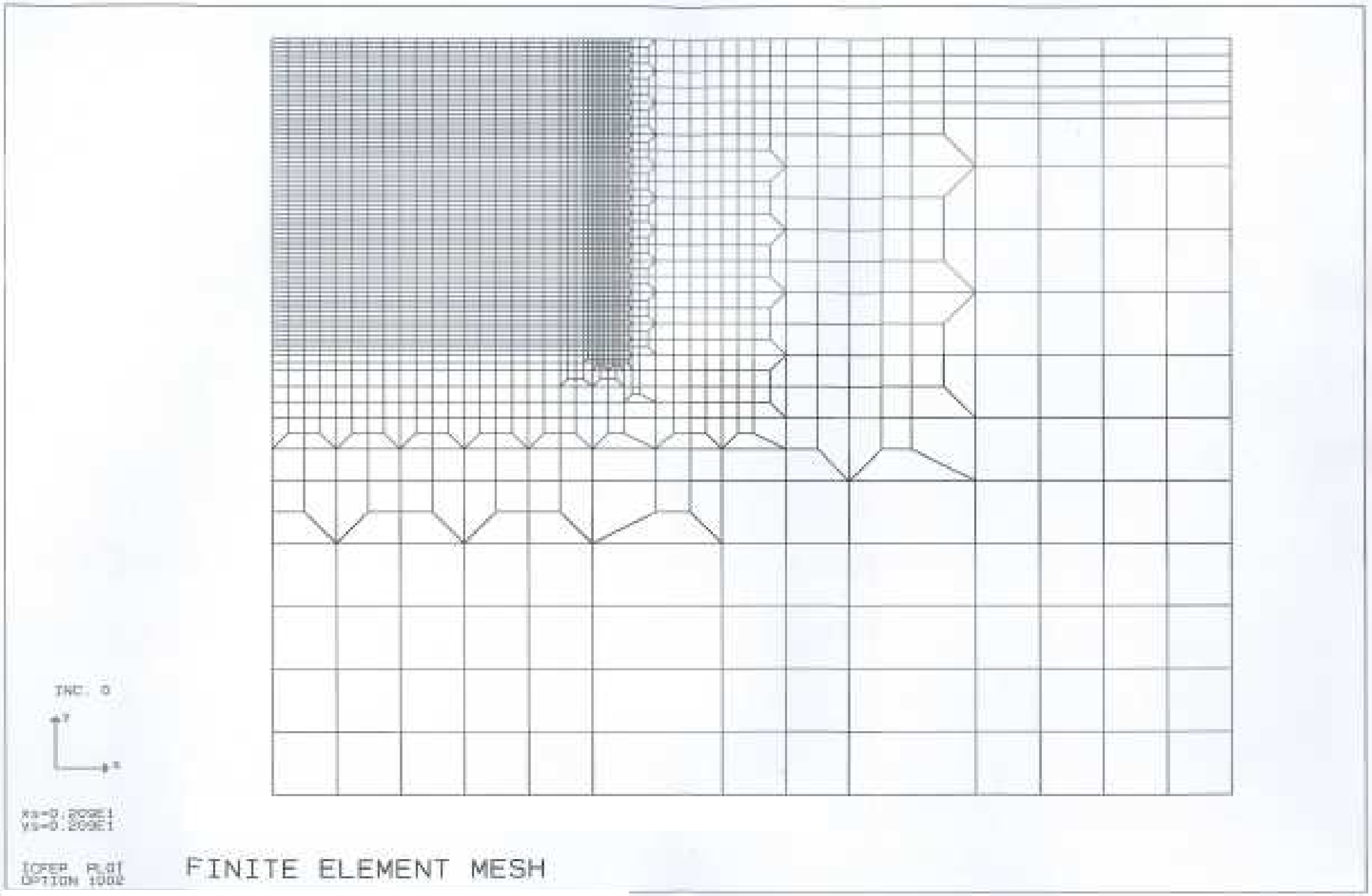


Figure 5.2. Finite element mesh.

5.1.2.- Finite element mesh:

The finite element mesh dimensions are 48m deep and 60m wide. This extension is considered enough for the modelling, relevant changes in variables are not influenced by the far boundaries. This condition is not required for the symmetry boundary.

The mesh is formed by 3317 elements which are expected to provide a discretisation fine enough, especially near the wall, the domain which accounts for bigger variation in the variables of the problem.

The following features of the mesh should be noted:

It is a 2D mesh, a simplification from reality (3D), such that the plane strain assumption is involved.

It consists of eight-noded quadrilateral solid isoparametric elements. The mesh has to be provided continuity; a corner of an element is a corner for all elements where this point is taken into account.

The parts of the domain where more strain is expected to occur have been modelled with smaller elements (0.25mx0.25m). Regions where variations of the variables of the problem are not as significant have been taken into account with bigger elements (up to 4mx8m). Quadrilateral elements are appropriate as long as the ratio between their dimensions does not exceed 1:10; this condition is respected in the entire domain. Also the different materials wished to be simulated have to be represented in different finite elements.

Special attention has been put in the definition of the elements in the domain which can be simulated to be excavated. E. Blackwell (2010) established tolerances of 0.5m, for a similar geometry and problem, which resulted insufficient for the appropriate characterisation of the results with the wished precision in some cases. Consequently 0.25 m spacing defined.

For the first 5m down the ground surface the maximum spacing of the elements is 1m in order to be able to model water tables at every meter depth.

5.1.3.- Boundary conditions

The bottom boundary is fixed in horizontal and vertical directions.

The right and left boundaries are fixed in horizontal direction.

A cantilever wall has been considered, the constraints for the displacement of the wall are the compatibility conditions of the system, no condition is directly imposed on it.

No hydraulic boundary conditions apply, the analyses proposed are assuming dry sand (or respecting pure drained conditions).

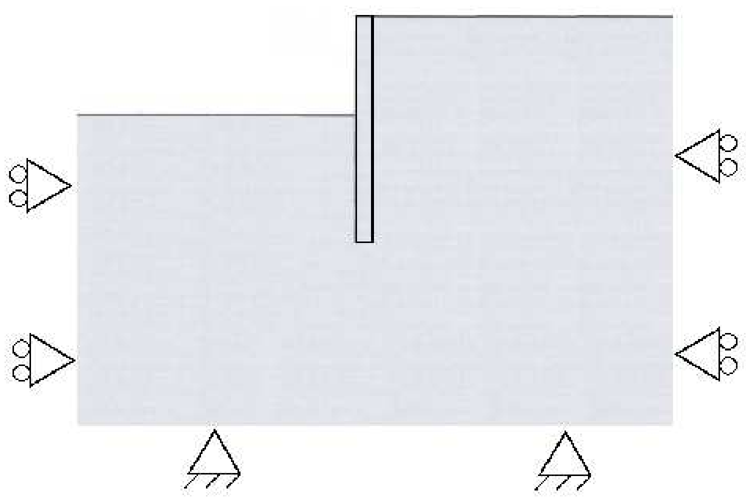


Figure 5.3 . Scheme of the boundary conditions of the problem.

5.1.4.- Initial conditions

The bulk unit weight assumed for the sand is 20kN/m^3, to lead to the corresponding initial vertical stress.

The value of K_0 is evaluated according to the Jaky formula (1948); $K_0 = 1 - \sin(\phi)$. The variations in the dilation angle impose different K_0 values in the initial conditions. In fact the initial conditions are a variation for the different cases for which is wished to be studied. However they are related to the value of the parameter ϕ either its characteristic or design value.

5.1.5.- Material parameters

Soil parameters

The soil is assumed to behave as elastic perfectly plastic, defined by linear material properties and the Mohr – Coulomb constitutive model (ICFEP model 16).

In all the analyses the value of the Young Modulus is set to $1 \cdot 10^5$ KN/m^2 and the poison ratio to 0.2. However the parameters defining the Mohr – Coulomb failure criteria vary for different analysis, except cohesion considered zero in all cases.

Wall parameters

The wall is assumed to behave as isotropic linear elastic, parameters: Young Modulus is set to $30 \cdot 10^6$ KN/m^2 (30 GPa) and Poïson ratio to 0.15.

Soil – structure interaction

All the elements used in the mesh are solid elements, no interface elements have been introduced. Consequently the properties in the interface result from the properties of the soil and wall, so the friction angle in the interface is the full friction angle of the soil.

5.2.- Investigation and study cases

The focus in this project is put on the comparison of the alternatives available for the factorisation of strength parameters and in assessing the effects of the dilation angle and its factorisation. The choice of the ϕ values for simulations {25, 30, 40} is arbitrary, but wishing to represent a frequent wide range of cases. Analyses are distinguished between zero and full dilation.

Especial attention is put in the behaviour at failure. It should be noted that the wall is modelled as linear elastic material, and it can be thought of an effectively rigid wall due to its width and significantly higher stiffness than the soil. No hinges will appear in the modelled wall under any situation, but it is allowed to bend. This provokes that the failure appears in the soil. This can be an acceptable assumption because one can think that the structural design (which is not the object of the investigation) can be always modified such that no plastic hinges appear in the wall.

In addition, the role of initial stresses is to be investigated. They are calculated imposing $K_0 = 1 - \sin(\phi)$. This leads to two alternatives to be investigated, ϕ taken as design value of the friction angle or otherwise the characteristic value. It should be noted that the input of AP1 will be always the characteristic, while for AP2 both apply.

The construction sequence to be simulated:

- Establish initial stress and boundary conditions
- Excavate step by step to the required depth
- For simulations following AP1, perform c'-ϕ' reduction to determine factor of safety.

The procedure for investigating the approaches close to failure involves firstly applying AP1 and obtaining an equivalent partial factor of safety; then studying AP2 with the parameters reduced by the calculated factor of safety from the beginning of the analysis.

With this methodology it is guaranteed that the parameters for the calculations relate to the design situation for which a ULS check would be relevant, i.e. determining if the reduced strength produces failure.

To assess failure, the main criterion has been looking at the “mechanism plot” –which can be found in the annex B for every calculation –. This mechanism plot is a representation of the incremental displacement vector over the whole domain of analysis for the last increment of the calculation (last conditions in which equilibrium was reached). The general appearance of this plot can indicate the formation of passive and active wedges, the kind of failure mechanism expected for this problem.

The main scenario proposed as object of the investigation is a 40m wide excavation in sand to 5m depth retaining wall 1m thick and 20m deep. A variation to 6.5m of the excavation depth is a complementary situation studied in order to enhance the validity of the comparison between the results for the different approaches (AP1, AP2) further than in a single scenario. Analyses for various values of the couple (ϕ, ν) are performed in every case.

The depth of the wall was fixed to 20m in the choice of the mesh. The presented study cases excavate up to 5 and 6.5m. The ratios excavated depth / depth of the wall is smaller than what a real design case would present but it allows investigating the behaviour for a wide nonlinear branch when increasing factor of safety.

6.- Results of the finite element method analyses

6.1.- Variables treated

This section aims to summarize the results of the analyses performed. Individual results for every of the calculations which are going to be referred can be found in Annex B. This includes plots obtained of the mechanisms of failure and the FOS-displacement curves with the computed points. However in this section, allowing for a clearer interpretation for the various curves per graph, the plots are presented in smoothed, continuous curves.

It has been considered relevant to include:

Last displaced shape of the wall which could be found in equilibrium, in a plot horizontal displacement (δ) along the depth of the wall. It is important to distinguish this from failure, where by definition, the displacements are infinite; consequently the numerical values of the results are dependent on the numerical scheme. However, when the cases of the parameter variations of the same excavation case are presented in the same graph; this allows comparing between them with the objective of being able to judge if the behaviour of the wall in terms of displacement shape is the same in all cases.

Evolution of the horizontal displacement at the top of the wall (also denoted δ as being an horizontal displacement) with the variation of the factor of safety, which allows appreciating the nonlinear behaviour of the system (AP1 simulations).

Both plots include horizontal displacements at the top of the wall. However, one can notice that the values shown for the same analysis do not always coincide. This is due to the increments/phases of increment in which have been taken. Information for the factor of safety can be acceded for some phases but this is not possible for the wall displacements which require having saved the information at the end of an increment of computation.

Bending moments of the wall in the last stage computed. This corresponds to the most unfavourable situation and can be associated with the bending moments when failure in the ground is reached.

In the presentation of results, it should be noted that f and ϕ denote the friction angle, while v denotes the dilation angle. For identifying the cases easily, the notation (ϕ, v) has been introduced, so (25,25) is referring to a case with 25º friction angle and 25º dilation angle.

6.2.- 5m excavation in dry soil

Cases analysed:

AP1	Characteristic values, then strength reduction		
ϕ	v	F.O.S.	ϕ failure
25	0	2.78	9.5
25	25	2.8	9.5
30	0	3.48	9.4
30	30	3.5	9.4
40	0	4.86	9.8
40	40	5.06	9.4
AP2	Factored input values		
ϕ	v	K0 (ϕ)	Failure
9.8	0	0.83 (9.8º)	Not complete, but very large displacements presented.
9.4	9.4	0.83 (9.4º)	Complete mechanism.
9.8	0	0.58 (25º)	*
9.4	9.4	0.58 (25º)	*
9.8	0	0.5 (30º)	*
9.4	9.4	0.5 (30º)	*
9.8	0	0.36 (40º)	*
9.4	9.4	0.36 (40º)	*

Table 6.1. Parameters input of each calculation and relevant scalar results.

*The conditions on initial stresses are incompatible with yield conditions.

The results of this set of simulations, as expressed in the table 6.1, in the figures below, and completed by Appendix B, confirm that the mechanism of failure in the soil is an active wedge in the extrados together with a passive wedge located at the intrados. In addition, for the cases with higher friction angles, the factor of safety is higher. The behaviour of the wall is very similar for the cases computed, presenting horizontal displacement along the depth. This displacement can be described in the majority of the cases as the combination of bending and a rotation around a point in the lower part of the wall. However, for the case (40, 0) its movement can be better associated with horizontal translation and bending; better acknowledged in figure 6.3. This behaviour imposes a distinction of this case from the rest, which might be related with an anomaly in the initial stages as shown in the slope of the correspondent curve (see figures 6.1, 6.4).

This anomaly can be thought of associated with the unstable behaviour around the perfect plasticity solution exhibited in numerical simulations of models with non-associated plasticity in which the plastic volumetric strains are forced to differ significantly (by means of imposing a high difference between ϕ and ν e.g. (40,0)) from the associated plasticity.

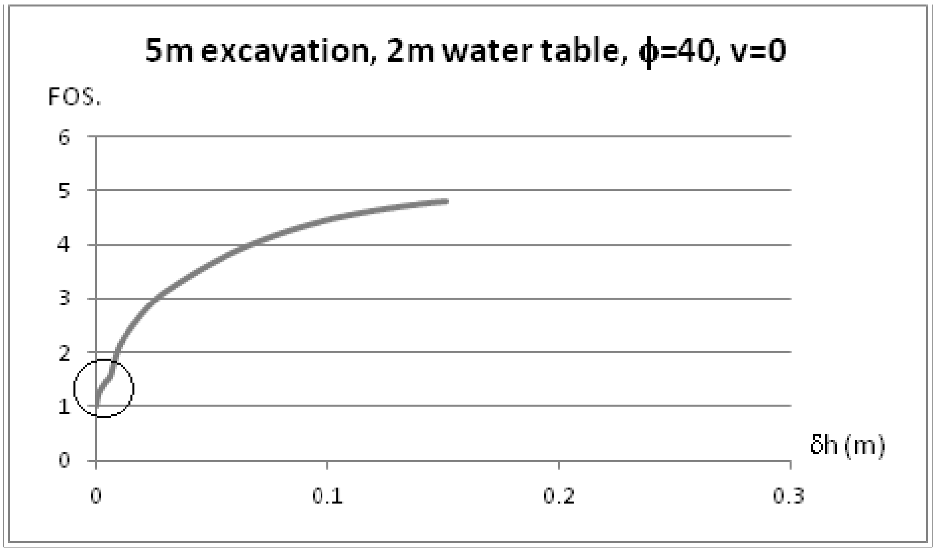


Figure 6.1 Nonlinear behaviour during strength reduction for the (40,0) case.

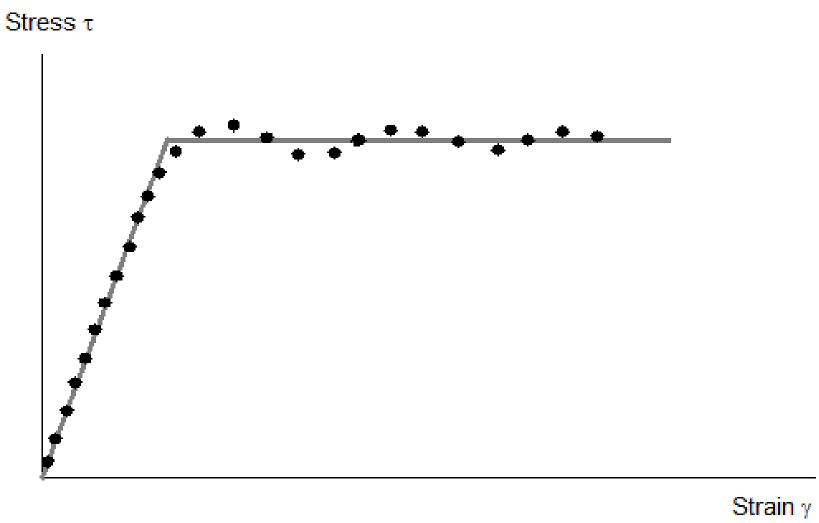


Figure 6.2 Conceptualisation of the phenomenon "unstable behaviour around the perfect plasticity solution exhibited in numerical simulations of models in non-associated plasticity in which the plastic volumetric strains are forced to differ from the associated plasticity". The figure aims to represent what the numerical model would predict from a Stress – strain curve for a point of the domain which reaches plastification.

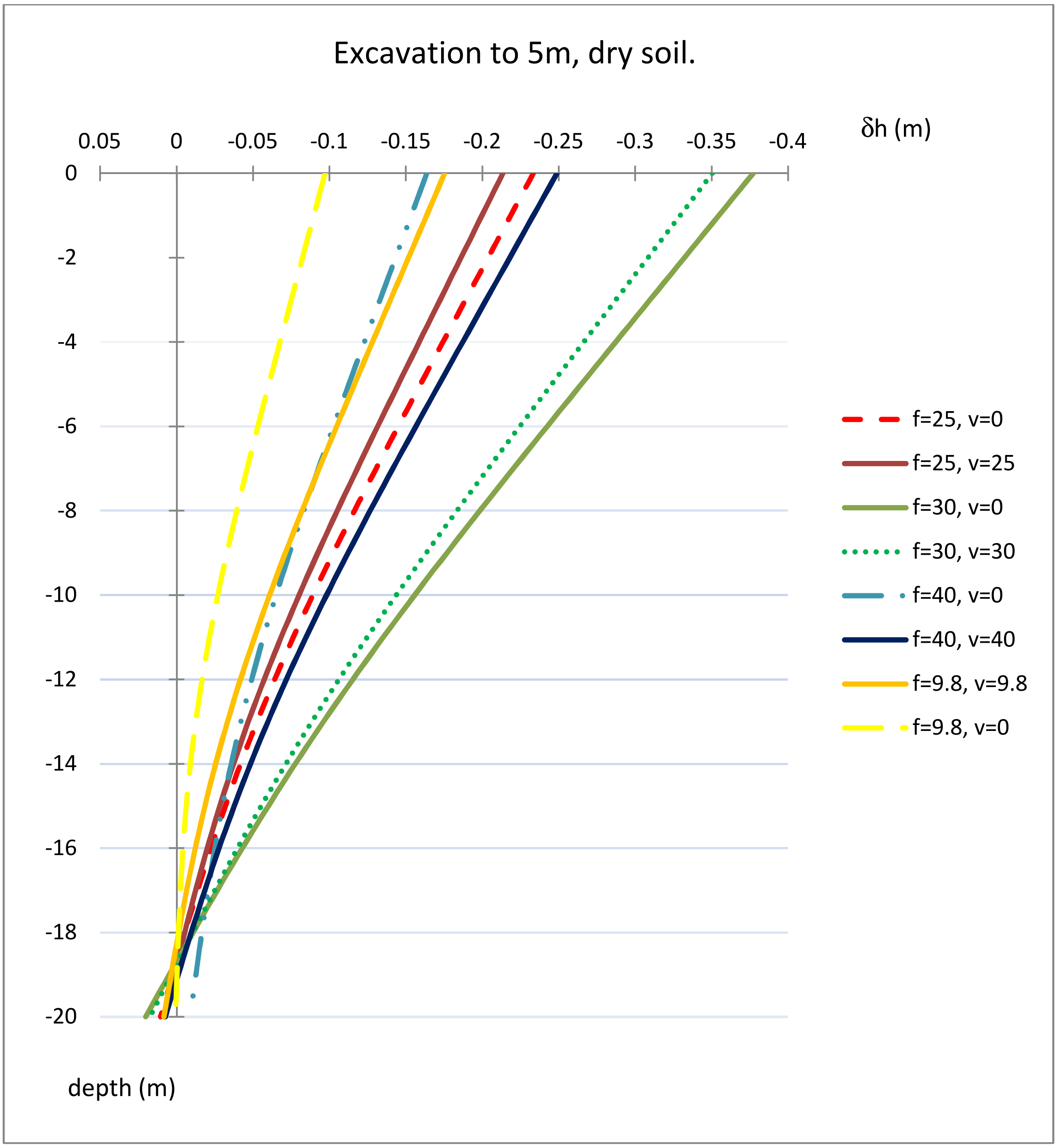


Figure 6.3. Displaced shape of the wall in the last stage of the calculation with equilibrium.

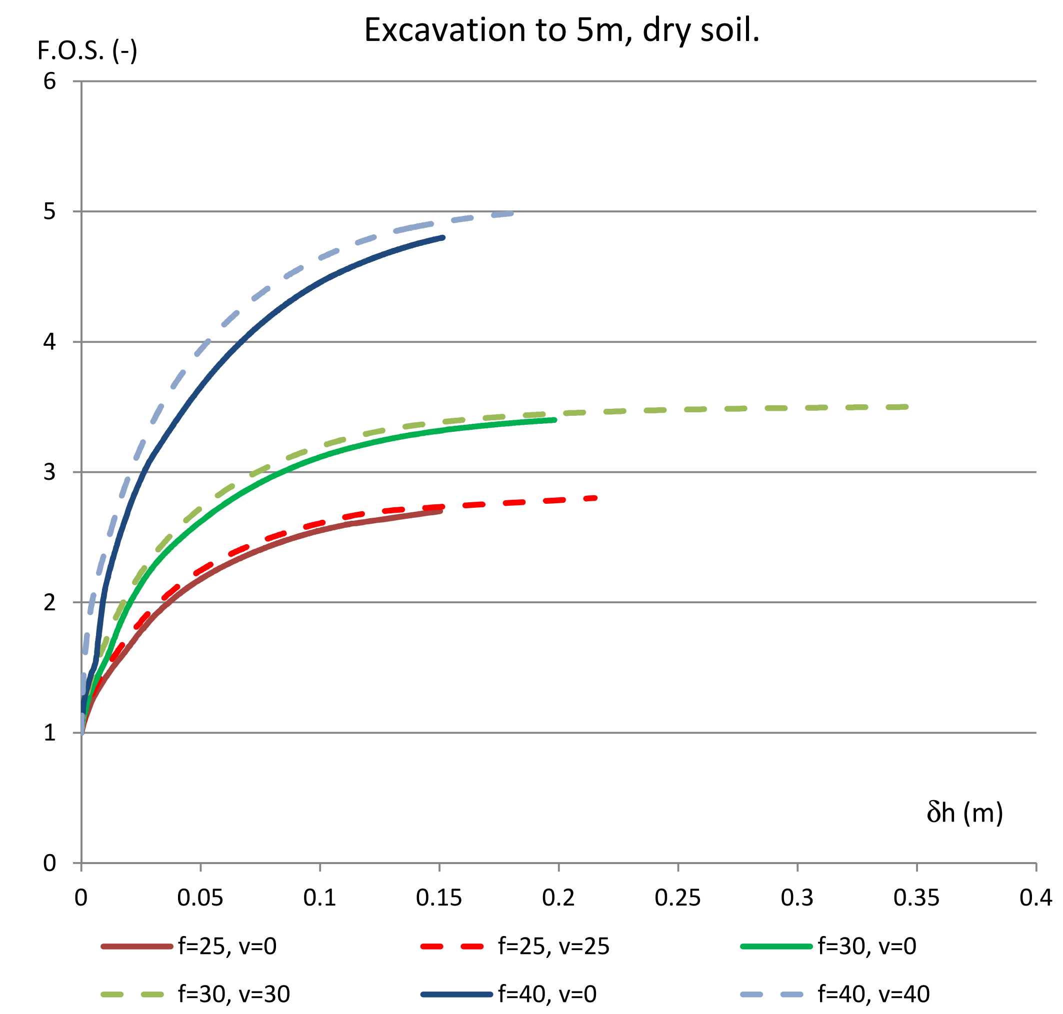


Figure 6.4. Nonlinear behaviour during strength reduction.

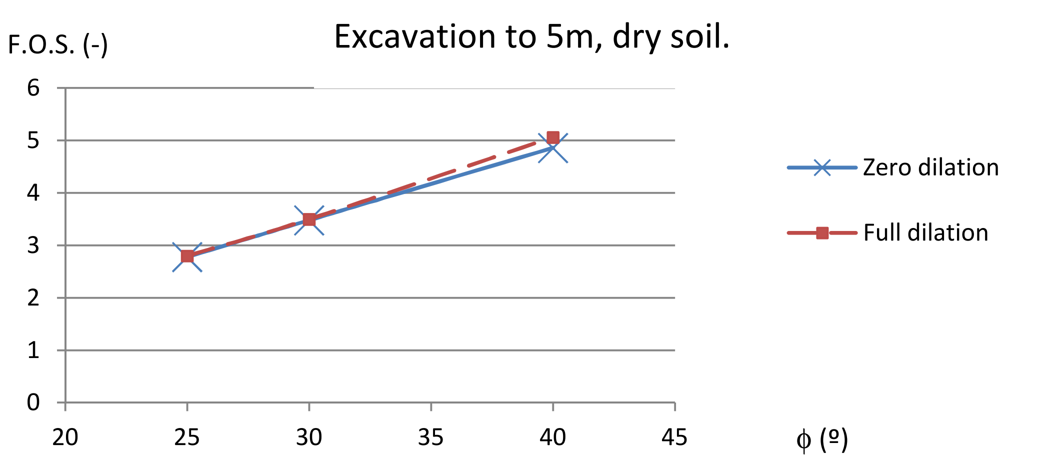


Figure 6.5. Factor of safety variation with dilation angle.

For the simulations in AP1 it is important to mention the strong trend shown in the computation of ϕ failure, the friction angle reduced by the corresponding factor of safety at failure (Table 6.1):

$$\tan(\varphi_{failure}) = \frac{tan\ (\varphi_{characteristic})}{F.O.S._{failure}}$$

The fact that simulations with different characteristic strength parameters and initial conditions result in the same friction angle at failure can be interpreted as these parameters having little influence on the problem at failure, and consequently associating an excavation depth with a required strength parameter. This occurs independently from the value of the dilation angle for this cantilever wall.

In contrast the effect of dilation results in slightly larger passive wedges mobilised (see mechanism plots in appendix B) and very slightly higher factors of safety for the same friction angle (Figure 6.5), this last feature being more significant for higher friction angles. However, this should not make any important difference for design.

It should be noted than from Figure 6.5 one can think of a linear relationship between the factor of safety and the characteristic strength.

Calculations with AP2 were not possible for initial stresses associated with characteristic values of the friction angle; the condition imposed by the yield criterion required higher values of K_0. This is a consequence of the problem leading to a high factor of safety so that $\phi_{failure}$ is considerably smaller than the characteristic. Consequently results from AP2 account for less realistic initial stress conditions.

Calculations with AP1 take initial stresses from the characteristic value of ϕ which is the value of the parameter at the start of the simulation. Then the strength reduction starts. It is important to notice that the computation of the non-linear range has to be made factor of safety controlled rather than displacement controlled (algorithm) but small increments (under the 0.02 used) may lead to big differences in displacement at a state close to failure. The accuracy and precision of the calculation is guaranteed by a sub-stepping of the factor of safety increment. Computations with full dilation resulted easily captured in the nonlinear range.

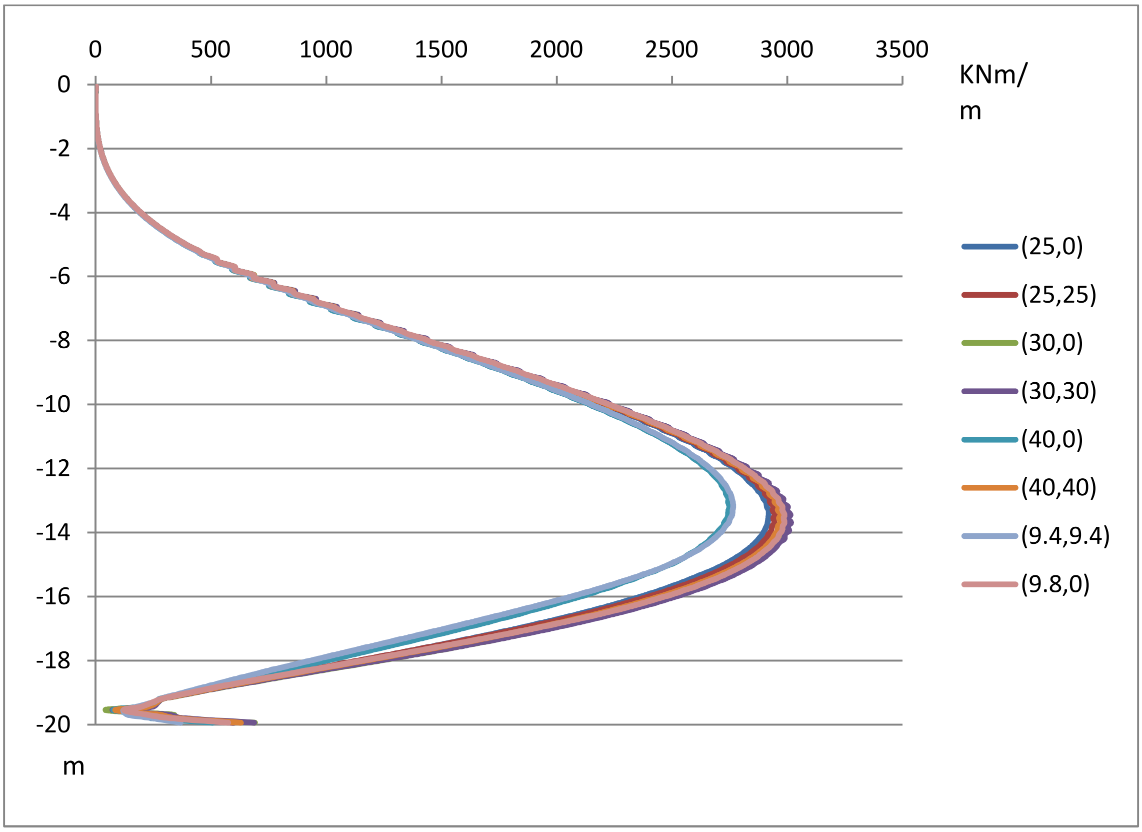


Figure 6.6. Bending moment distribution for the cases computed.

The most important feature that can be appreciated from the figure above is the similarity between the majority of bending moment distributions, both in shape and values along the domain. This can be explained because the plot corresponds to the stage for which a statically determinate system is failing. Specifically one the two plots which differ from the other ones is the (40,0) case, which anomaly has been explained. The (9.4, 9.4) also seem to differ with no apparent immediate reason. From this similarities one can reason that, for every depth the difference between the active and passive pressure at failure is very similar, which is the only way to lead to the same bending moment distribution (neglecting the contribution of the vertical friction times the lever arm of the width of the wall). Finally, the shape of the bending moment distribution is the typical shape for this kind of wall except for the values shown in the lower 0.5m. In this domain reality is not well represented by the simulation. It should be showing tendency to zero, what should be obtained in an idealised moment-free end. In this domain the displacement field is complex, with small values in comparison to the rest of the domain (point of rotation of the wall). Values reach over 1/6 of the maximum bending moment; they are not a real result. However this does not bring any difference for design.

From a wider perspective, comparing cases computed with AP2 with those computed with AP1 cannot definitely establish whether any of the methods is more conservative than the other one. In AP1 failure is forced to occur, meaning that the identification of the point “just” at which not being able to reach equilibrium is very well defined. However in AP2 failure is not imposed but assessed from the results (mechanism plots, convergency). When for the last step of excavation equilibrium could not be found -case 5m (9.4, 9.4)- it clearly indicates that failure has been reached. In other cases -(9.8, 0)- it can be found that the last incremental displacement of excavation is significantly big and relatively much larger than the previous. Then it could be proposed that it might be a matter of the discretisation of the increment in excavation that failure has not been identified as long as the mechanism is almost complete and in a subsequent small step (excavation to h +dh) equilibrium would not be reached. However, as equilibrium has been reached, one cannot deny that the limit state has been satisfied.

One should notice that in this last case the ϕ parameter has been chosen slightly superior (9.8, as the maximum of the values of the trend) rather than directly corresponding to the case (9.5) and AP2 has successfully identified the correspondent over-strength completing the whole simulation.

Consequently one could state that both procedures are leading to the same design decision. Using AP1 results in a more transparent procedure quantifying the margin of security. On the other hand, AP2 permits to use conventional commertial software to apply the material factoring approach without need of discussing which algorithm should be implemented for strength reduction.

Regarding the bending moment distribution one can think in the same direction, both procedures lead to determine the same design values, the slight differences of results for bending moments do not coincide with the implementation of AP1 or AP2, but results with the different approaches tend to coincide.

As an indication of the comparative computational cost, having reached skill in the operation of the program, a simulation with an approach AP2 could be completed within one day, while the completion of simulations with AP1 was in the order of one week (6days). (ICFEP as implemented in IC Geotechnical department using “Jenkins” Server).

6.3.- 6.5m excavation, dry soil.

Cases analysed:

AP1			
ϕ	ν	F.O.S.	ϕ failure
25	0	1.96	13.4
25	25	2.04	12.9
30	0	2.46	13.2
30	30	2.54	12.8
40	0	3.58	13.2
40	40	3.68	12.8
AP2			
ϕ	ν	K0	Failure
13	13	0.772	YES
13.2	0	0.658	*
13	13	0.658	*
13.2	0	0.5	*
13	13	0.5	*
13.2	0	0.357	*
13	13	0.357	*

Table 6.2. Parameters input of each calculation and relevant scalar results.

*The conditions on initial stresses are incompatible with yield conditions.

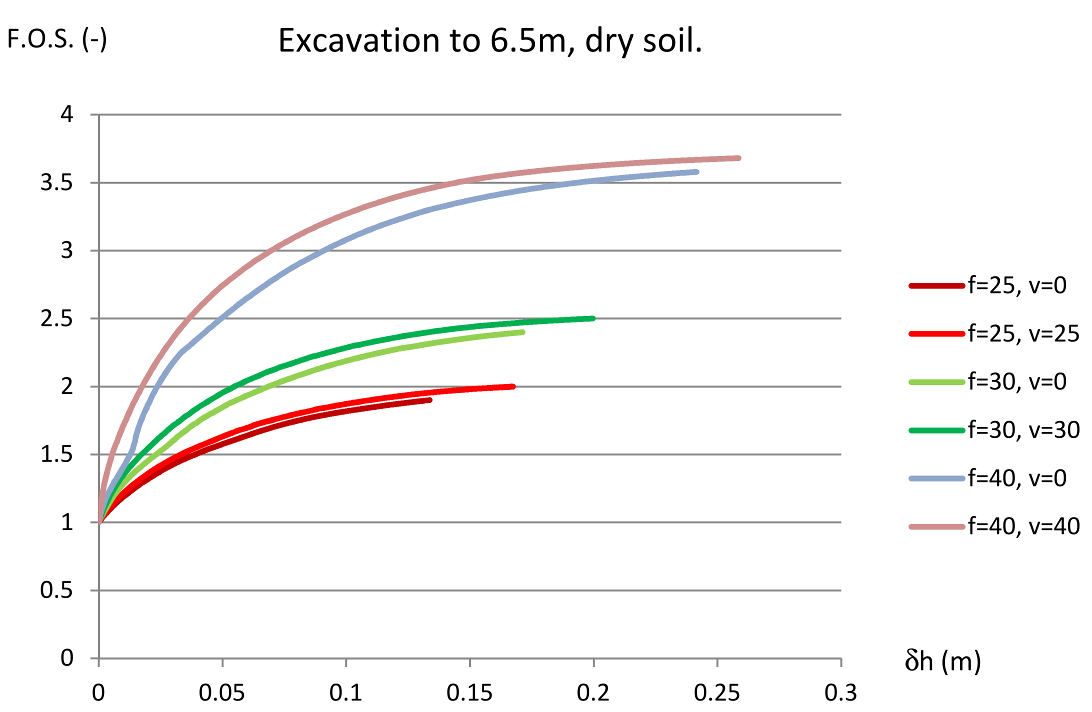
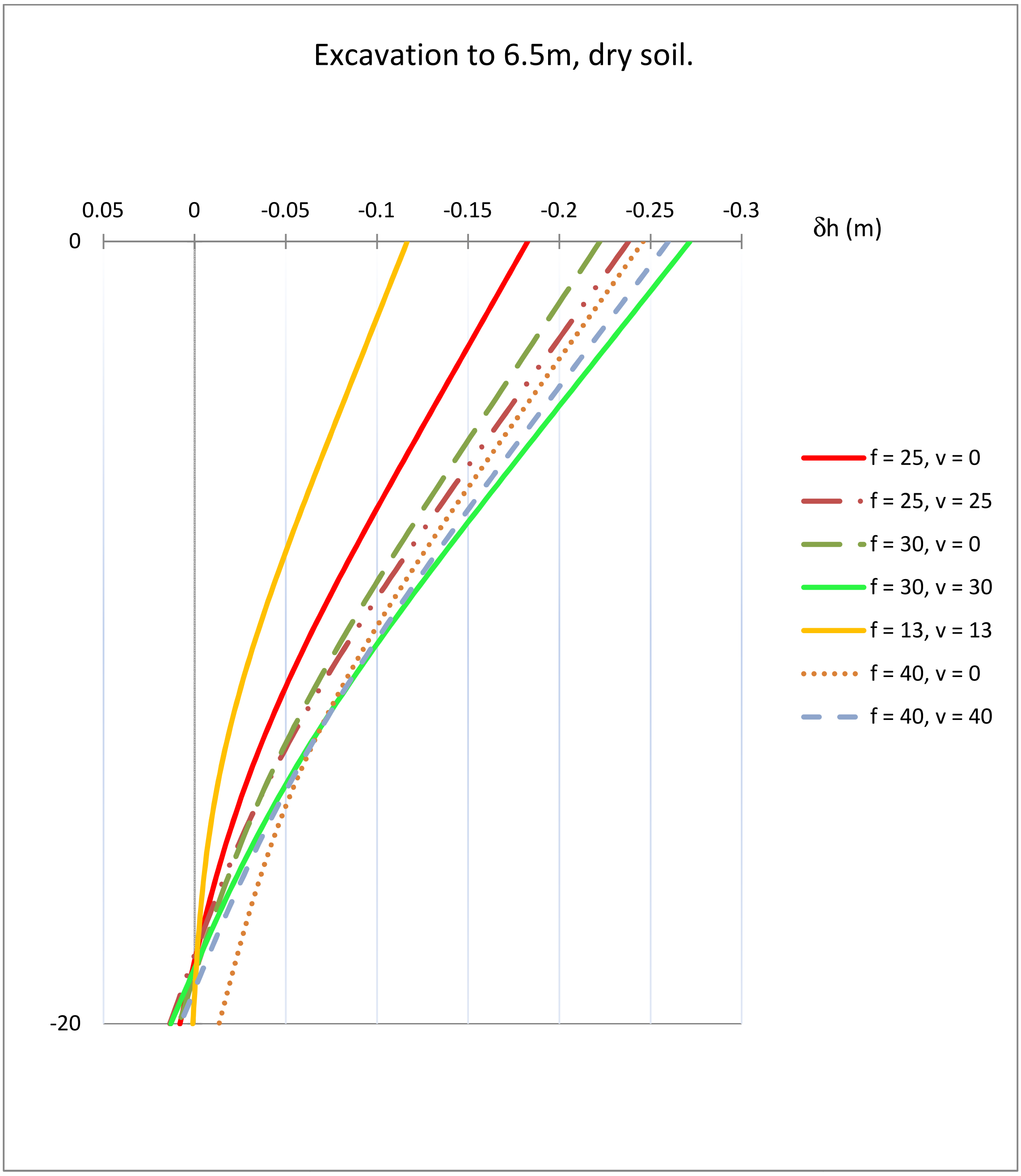


Figure 6.7 . Nonlinear behavior during strength reduction.

The idea of calculating a case for 6.5m excavation was a consequence of not finding any case in the 5m excavation of a compatible initial stress with the yield conditions. For a 6.5m excavation the factor of safety is lower, so the friction angle at failure (for AP1) is higher, and consequently there is less difference between the characteristic and reduced parameters. However, again any of the initial stresses corresponding to the characteristic parameters are compatible with the yield conditions. One should bear in mind that this design situation is more similar to what one would find in reality, but also allows for a smaller evolution of the nonlinear range to be investigated while performing strength reduction.



The cases computed in this 6.5m excavation simulations present a general behaviour similar to the previous 5m excavation simulations; the wall bends and rotates about a point near its bottom. (40,0) presents the same anomaly. As well the cases in AP1 still show a marked trend in $\phi_{failure}$ convergence.

In this set of simulations the calculations with full dilation are again showing bigger wedges mobilised at failure (than the same cases with zero dilation). Similarly, the computations reached failure at higher displacements and with slightly higher factors of safety – this last tendency more marked for higher values of ϕ - . However, for the later consideration it can be stated that the friction angle is clearly the parameter which controls the factor of safety, as expected.

In the bending moment distribution for this 6.5m excavation, the different cases show a strong trend regarding the shape of the distribution and the values along the domain. However, the cases don't seem as coincident in the maximum value in comparison with the cases for 5m excavation, but the range of difference between the extremes is approximately the same (8.8% vs. 7.6%). Again the (30,30) reaches the maximum value but no trend can be inferred as long as the next values don't correspond. Again no significant difference can appreciated from cases computed with AP1 or AP2.

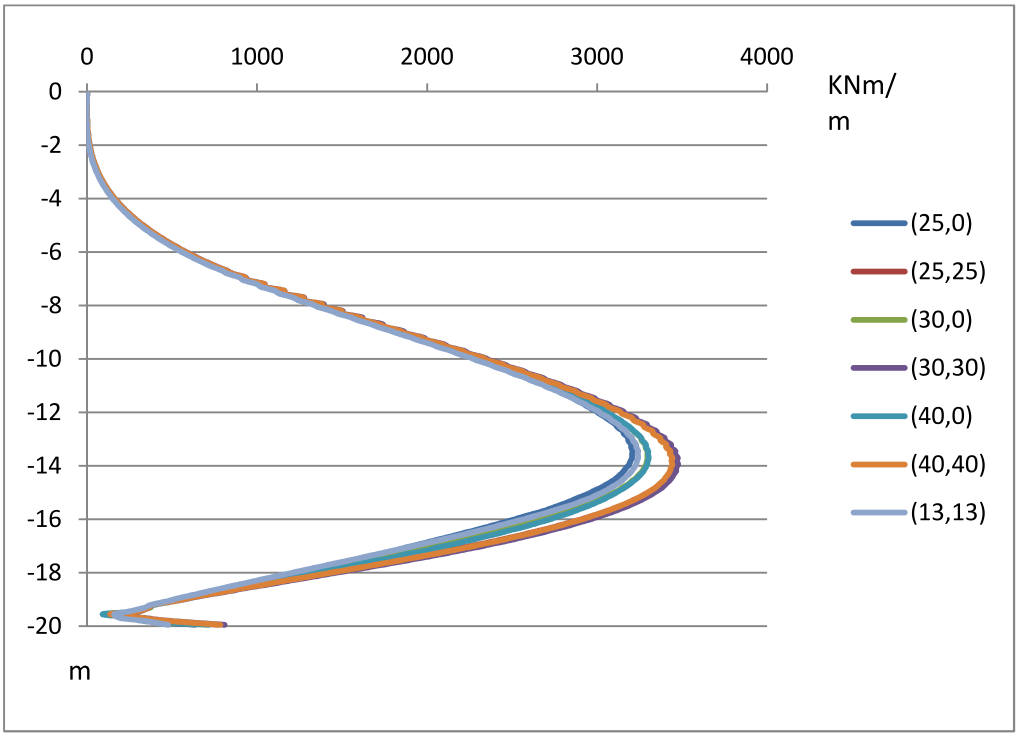


Figure 6.9 . Bending moment distribution for the cases computed till excavation 6.5m

6.4.- Validation of results

This section aims to show that the results of the finite element analysis are according to the experience accumulated and can be considered correct.

Despite the fact that F.E.M. is the only tool which can provide a result of the desired accuracy and precision for all the variables of interest in the whole domain; it is possible to asses if certain figures output of the analysis are acceptable with by the reliability of some results from other methods in predicting some variables.

In particular, regarding to bending moments:

It is generally considered valid that the part of the wall above excavation receives an horizontal pressure well modelled considering the soil in Rankine active state:

$$Taking\ into\ account\ c' = 0\ and\ \varphi\ as\ associated\ with\ failure, the\ horizontal\ pressure:$$

$$\sigma_H = Ka \cdot \sigma_V \quad ; \qquad Ka = tan^2\left(\frac{\pi}{4} - \frac{\varphi}{2}\right) \quad , \quad \sigma_V\ from\ vertical\ equilibrium\ \sigma_V = \gamma \cdot x$$

aKnown the lateral pressure distribution, as long as the wall is statically determinate it is possible to analytically compute (by simple mechanics) the bending moment in that domain. Hence, one could consider this a reliable check.

It has been considered:

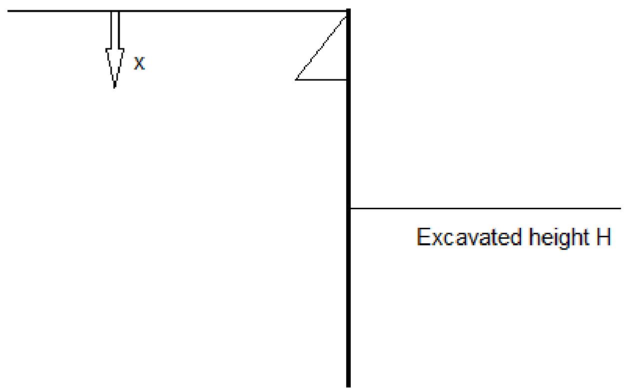


Figure 6.10. Scheme of the pressure distribution over the domain considered for computing the bending moment in the superior part of the wall.

So by introducing the formulae above and taking moments of the pressure distribution over depth "x":

$M\ (x) = \frac{1}{6} Ka \cdot \gamma \cdot x^3$ at $0 < x < H$

For the comparison of results it should be noted that:

- This formula is valid for both H = 5m and H = 6.5m, cases computed by FEM.

- The results in 6.2 and 6.3 show that the bending moments of the cases computed can be taken as coincident in all cases for the domain at which the formula above refers, so to compare them it has been chosen the obtained FEM bending moment as any of the cases computed. - H = 5m -> (40,40) and H = 6.5m (25,25) –

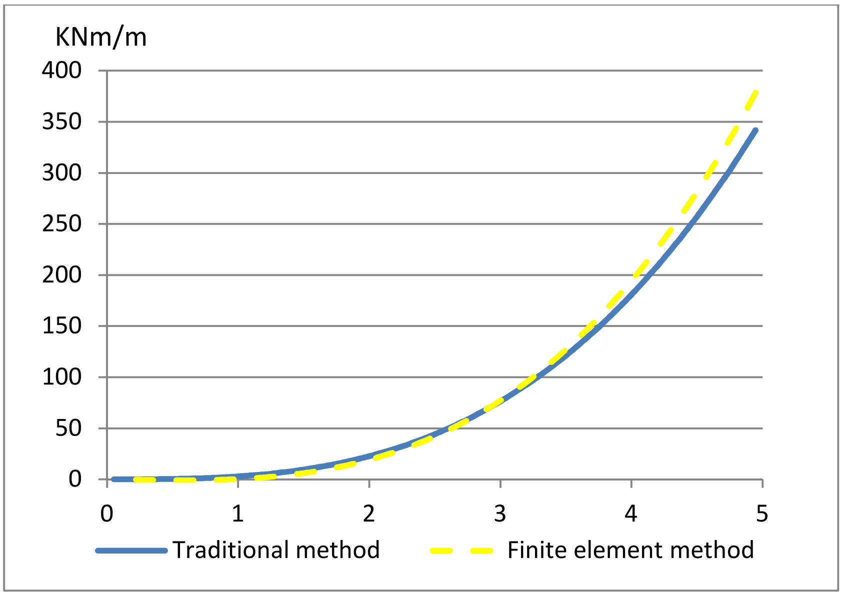


Figure 6.11. Bending moment distributions from FEM and Rankine, 5m excavation case.

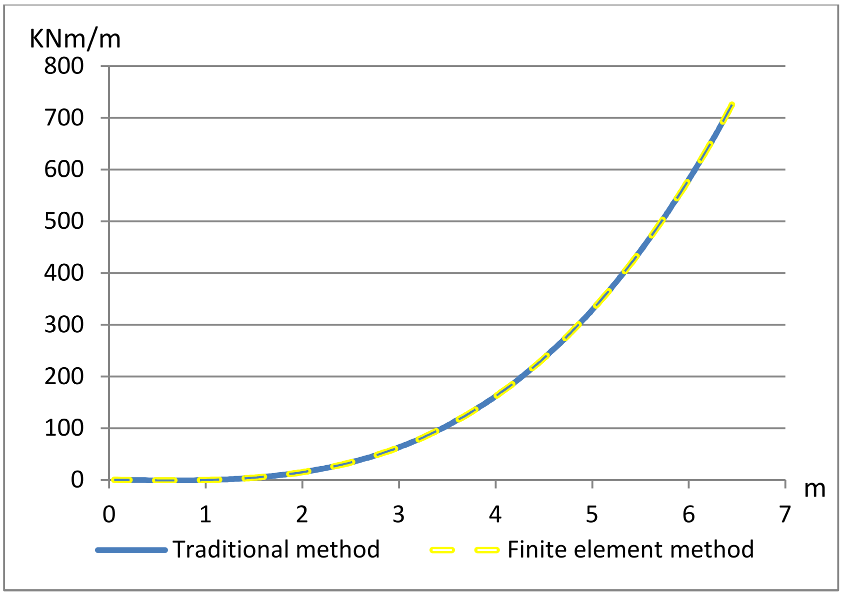


Figure 6.12. Bending moment distributions from FEM and Rankine, 6.5m excavation case.

From the results, it can be considered that the finite element analysis can be validated:

- The shape of the bending moment distribution corresponds to what is expected from experience and illustrated by the result of Rankine Theory.
- The order of magnitude of the values computed relates satisfactory with the figures expected.

7.- Conclusions

- Setting a nonzero value for dilation in this boundary value problem has a negligible effect in increasing the factor of safety, which is controlled by the friction angle. In cases computed with nonzero dilation incremental displacement plots show that bigger wedges are mobilised.

- The treatment of the dilation while performing strength reduction has been its factorisation maintaining proportion with the friction angle. The effects of the parameter have been shown to be accounted in the calculations, and the theoretical constraints respected. Therefore this can be considered a valid procedure.

- The stepped reduction strength procedure(approach AP1) implemented by introducing the factor of safety as a state parameter, leads to find a value of the required friction angle which is reasonable to consider not influenced by the initial conditions or its initial value as a strength parameter (characteristic value).

- The bending moment distribution at failure obtained for different cases computed with the same excavation level tends to be coincident independently from the friction angle (strength parameter), initial conditions and factorisation approach. This can be explained by the problem proposed being thought of statically determinate.

- The implementation of the initial factoring approach (AP2) with initial stresses corresponding to the characteristic value of the friction angle is not generally possible for values of the factor of safety which are usual in design of embedded walls. The conflict involves incompatibilities of the initial stresses with the yield function. Consequently, approach AP2 has to be usually implemented assuming less realistic initial stresses.

When the aim of the analysis is design, the final objective is to check a limit state and obtain a design bending moment distribution. Results obtained with AP1 and AP2 lead to the same decisions because for analogue cases, because it can be identified if a set of safety factors can be accounted in that design, and produce equivalent bending moment distributions.

Hence, the design of a cantilever embedded retaining wall under the conditions simulated can be undertaken without modifying any commercial software for implementing AP1. However, AP1 is a more transparent procedure accounting for a better modelling of the real phenomena, but results are computationally much more expensive, the approach is not extended in the common engineering practise, and even different software implement diverse algorithms, some of which are only valid for certain constitutive models.

7.1.- Recommendations for further work

This investigation was proposed by the Geotechnical Section of Imperial College as a starting point for the assessment on procedures of implementing partial safety factors in the analysis of retaining walls in sand. In turn, this piece is one of the cases for which research aims to establish guidance in geotechnical design undertaken by means of finite elements; being in accordance to the principles of Eurocode 7. Together with the conclusions, the value of this investigation would be to highlight which points should be revised and taken into account in order that the analyses presented in this work may certainly be improved and extended. ICFEP is a very powerful tool with which every parameter and procedure of the analysis (in particular the stepped strength reduction – AP1) can be controlled, but requires experience to deal with the implementations, which can get to be cumbersome if lacking of guidance. Particularly it is proposed to address the investigation considering:

- To change the geometry of the retaining wall:

  In thickness, 70-80cm can be considered enough for an embedded retaining wall, and more close to the reality of design.

  In depth (or similarly excavate to a deeper level): The usual design practise requires an smaller ratio total wall depth (D)/ excavation depth (h) than the cases investigated. Usually with D/h not higher than 3, the ultimate limit state can be satisfied in most of the conditions, so the design is economically motivated in that direction. For the purpose of the present investigation it can be justified that in those cases the implementation of AP1 leads to a smaller non-linear range (safety factor is lower). Posterior to this study one can state that the monitoring of the nonlinear range does not bring particularly interesting information for the design of the wall. What is more, a lower ratio (allowing for a smaller safety factor) would determine a higher strength parameter at failure from which the range of initial stresses permitted by the yield function would be more similar to the real initial stresses. That would enable more realistic analysis in terms of initial stresses for the reduced strength as an input (AP2).

- Avoid simulations in which plastic volumetric strains are forced to differ significantly from associated plasticity (e.g. by imposing a high difference between ϕ and ν) because this leads to unstable numerical behaviour.

- Include the water table: Excavations below the water table will require to model the problem with an additional surcharge equivalent to the pressure of the column of water at excavation level. This was a software-dependent feature which came across during this investigation. However it has been considered that there were not enough correct consistent results to illustrate any conclusion for this case. When performing these simulations it is convenient to assume pure drained conditions to avoid coupling the governing equations with consolidation. No temporal hydraulic boundary conditions apply, water is considered in equilibrium. Consequently monitoring the pressures in the domain making sure the hydrostatic distribution is maintained can be a helpful proceeding.
- Include interface elements between the soil and the wall, which would allow to analyse the results for different friction conditions, adapted to real cases (some of them different from $\delta = \phi$ as preliminarily taken in this investigation).
- Validate the results presented with a more advanced constitutive model.
- Simulate cases in which the conditions of the wall don't lead it to be statically determinate. The wall can be propped, anchored, fixed at the toe or at some level. This is a common engineering practise, and the conclusion over the bending moment distribution is very likely to be different than for the cantilevered case.
- Include surcharges in the domain of analysis: It should be noted that it would require a discussion on the load of the surcharge varying when performing strength reduction to account for its correspondent safety factor. Moreover, if coincident with the inclusion of water table with excavation level below, the surcharge correspondent to the water pressure should be treated differently (without factorisation).
- It may be of interest to extend the analysis to other kinds of soil, other soil conditions (layers) that would complete the range of cases for this boundary value problem. As well, to check that the hypothesis of plane strain is linked with the real phenomena and undertake investigations in which the mechanism of failure interacts with the symmetry boundary of the problem. And finally, to assess the hypothesis of a wall "wished in place" for performing simulations.

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Appendix

It has been considered relevant to include:

APPENDIX A – Eurocodes and design. Complement to section 3.

APPENDIX B – Results. Complements to section 6.

APPENDIX C – Explained example of code implemented.

Appendix A – Eurocodes and design

The Eurocodes and relationships between them:

EN 1990 – Eurocode: Bases of structural design

EN1991 – Eurocode 1: Actions on structures

EN 1992 – Eurocode 2: Design of concrete structures

EN 1993 – Eurocode 3: Design of steel structures

EN 1994 – Eurocode 4: Design of composite steel and concrete structures

EN 1995 – Eurocode 5: Design of timber structures

EN 1996 – Eurocode 6: Design of masonry structures

EN 1997 – Eurocode 7: Geotechnical design

EN 1998 – Eurocode 8: Design of structures for earthquake resistance

EN 1999 – Eurocode 9: Design of aluminium structures

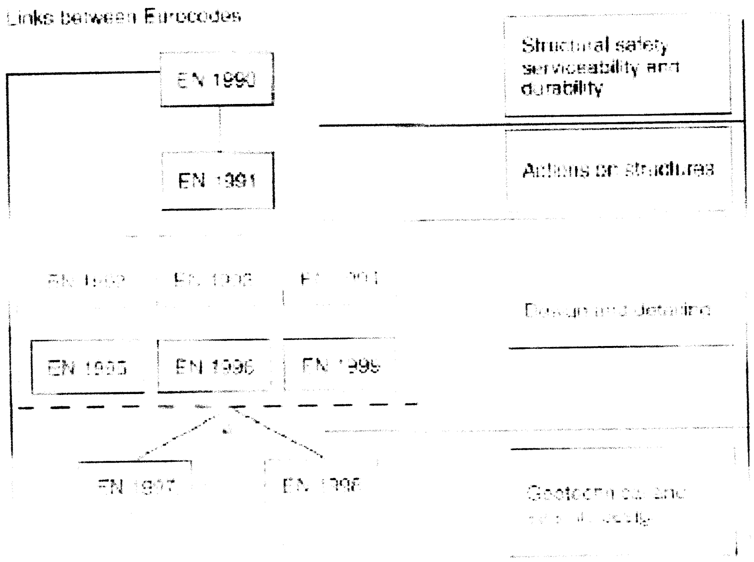


Figure A.1. Relationships between Eurocodes.From Frank et al. (2004).

Obtention of characteristic values.

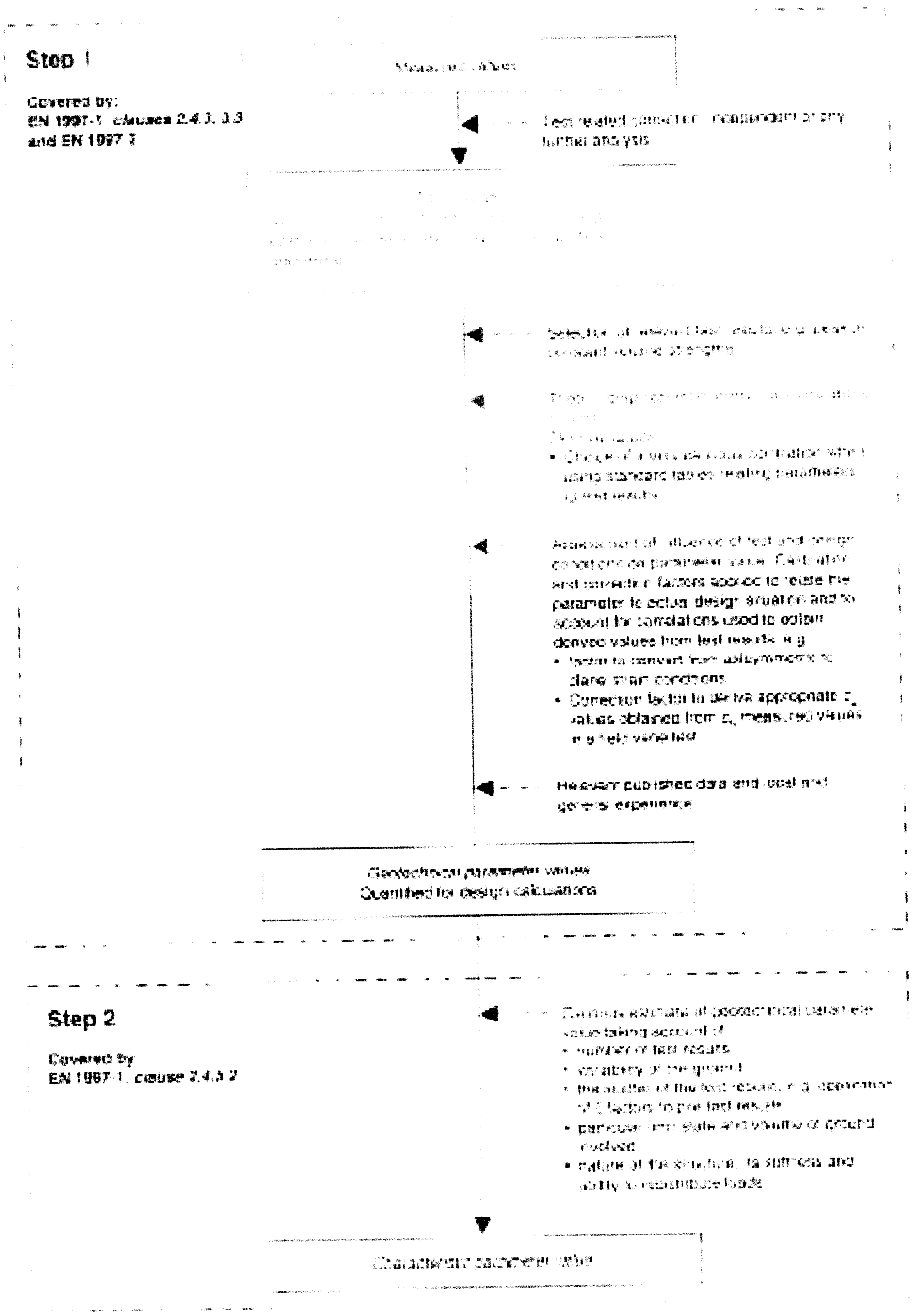


Figure A.2. Procedure for determining geotechnical parameters before factoring. From Frank et al. (2004).

Appendix B – Results

In this chapter the the relevant output of the calculations is presented in plot format.

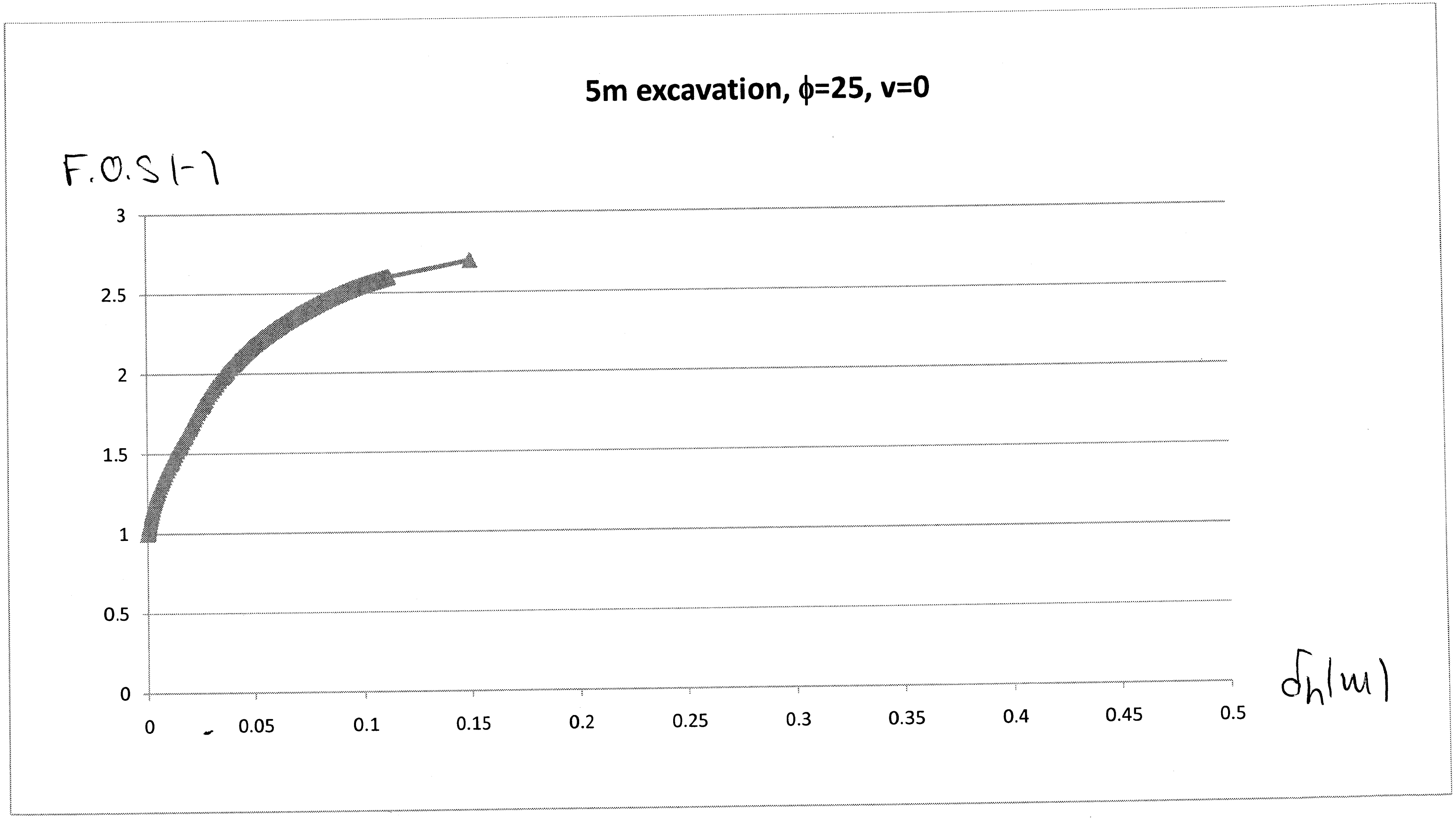
For calculations involving AP1:

- Mechanism shown as vectors of incremental displacement in the domain. This information is important to assess failure.
- Plot of factor of safety (F.O.S.) against displacement at the top of the wall (δ). This information is relevant to assess if the calculation has properly entered in the non-linear range. In various plots it attention has been put in representing the change in slope of the curve rather than trying that the plotted domain corresponds to the majority of the area. Furthermore, similar axes along the cases have been employed to easily compare between the plots.

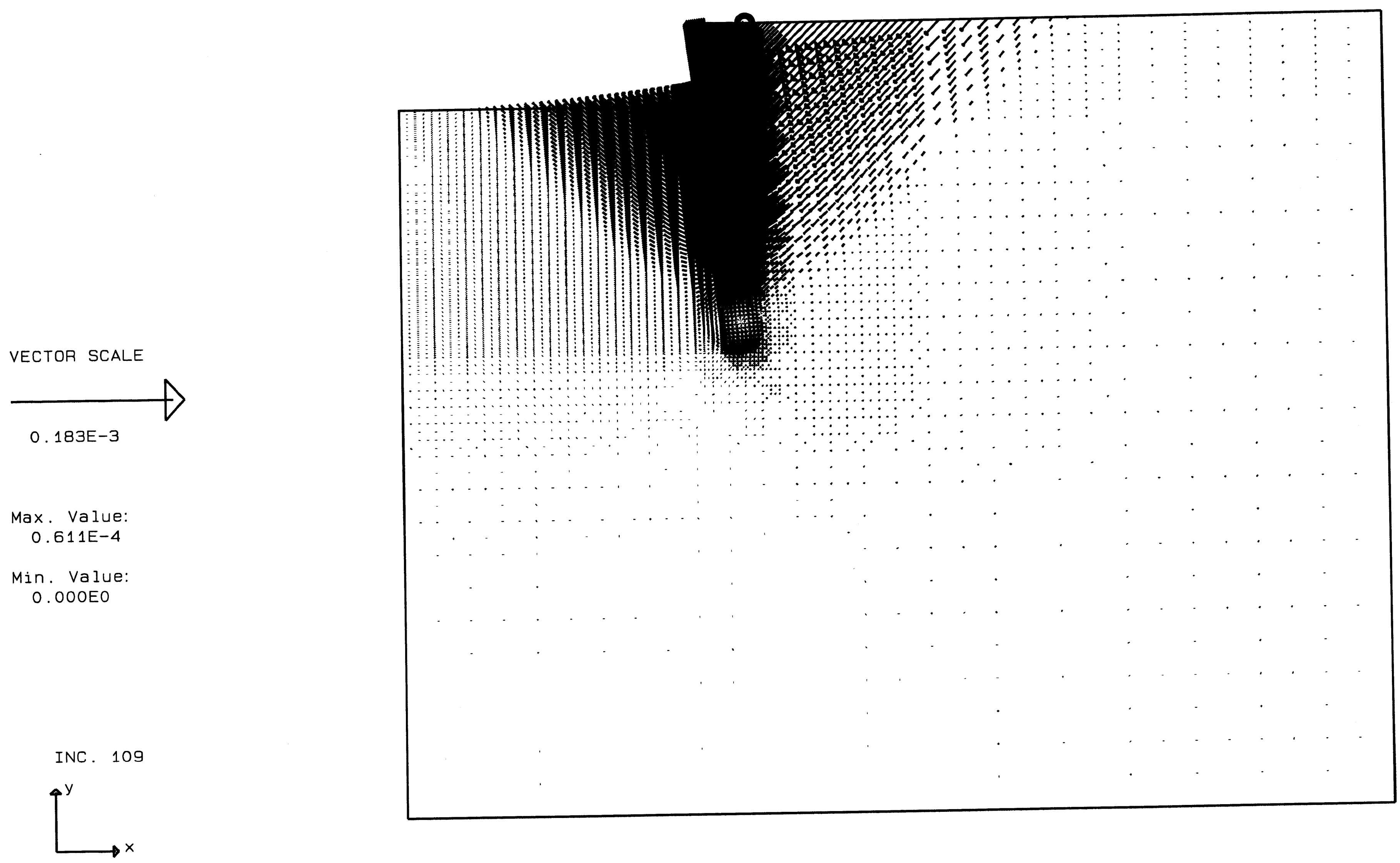
For calculations involving AP2:

- Mechanism shown as vectors of incremental displacement in the domain. This information is important to assess failure.

Cases presented in order of appearance in the documents.

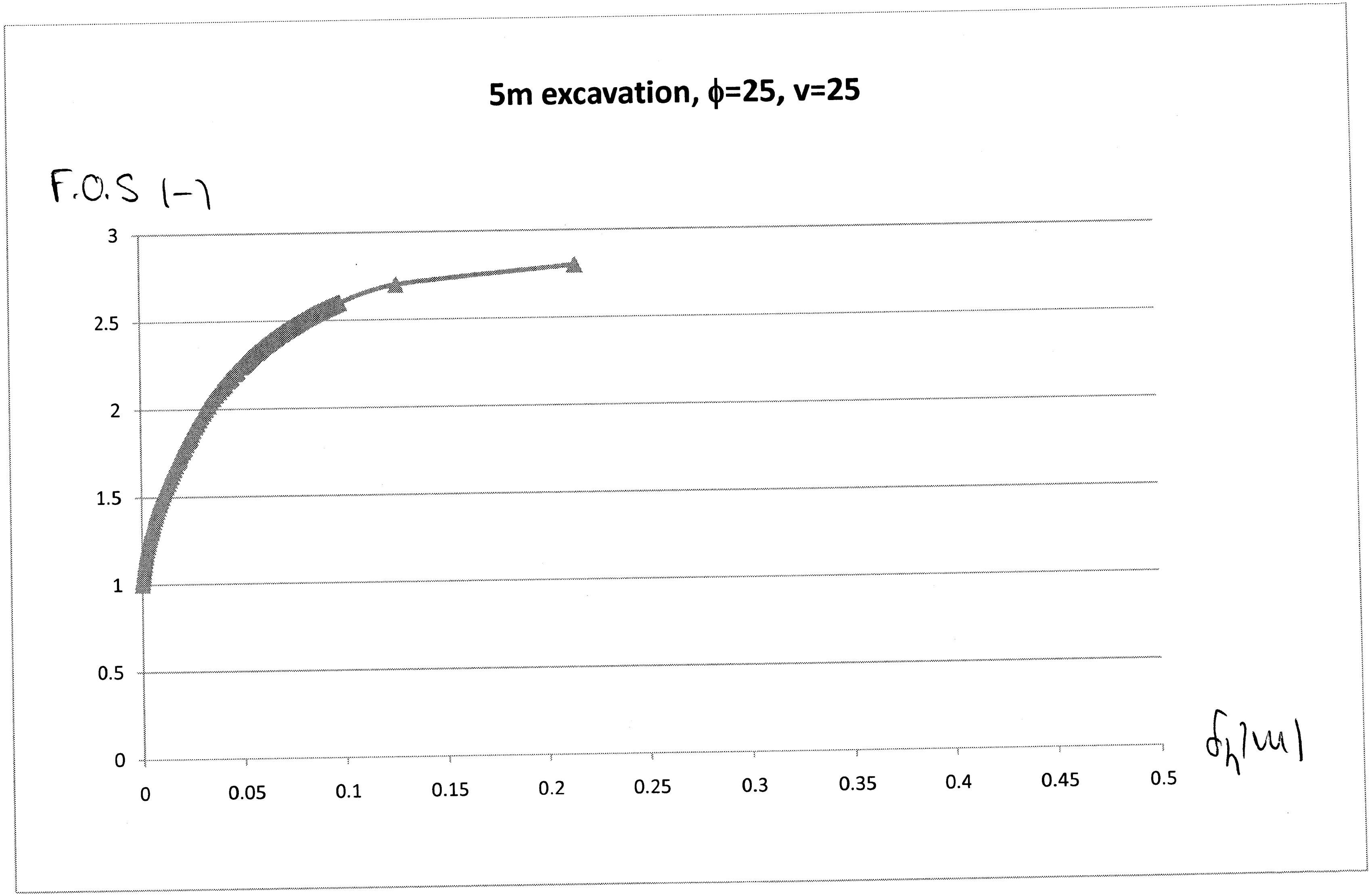


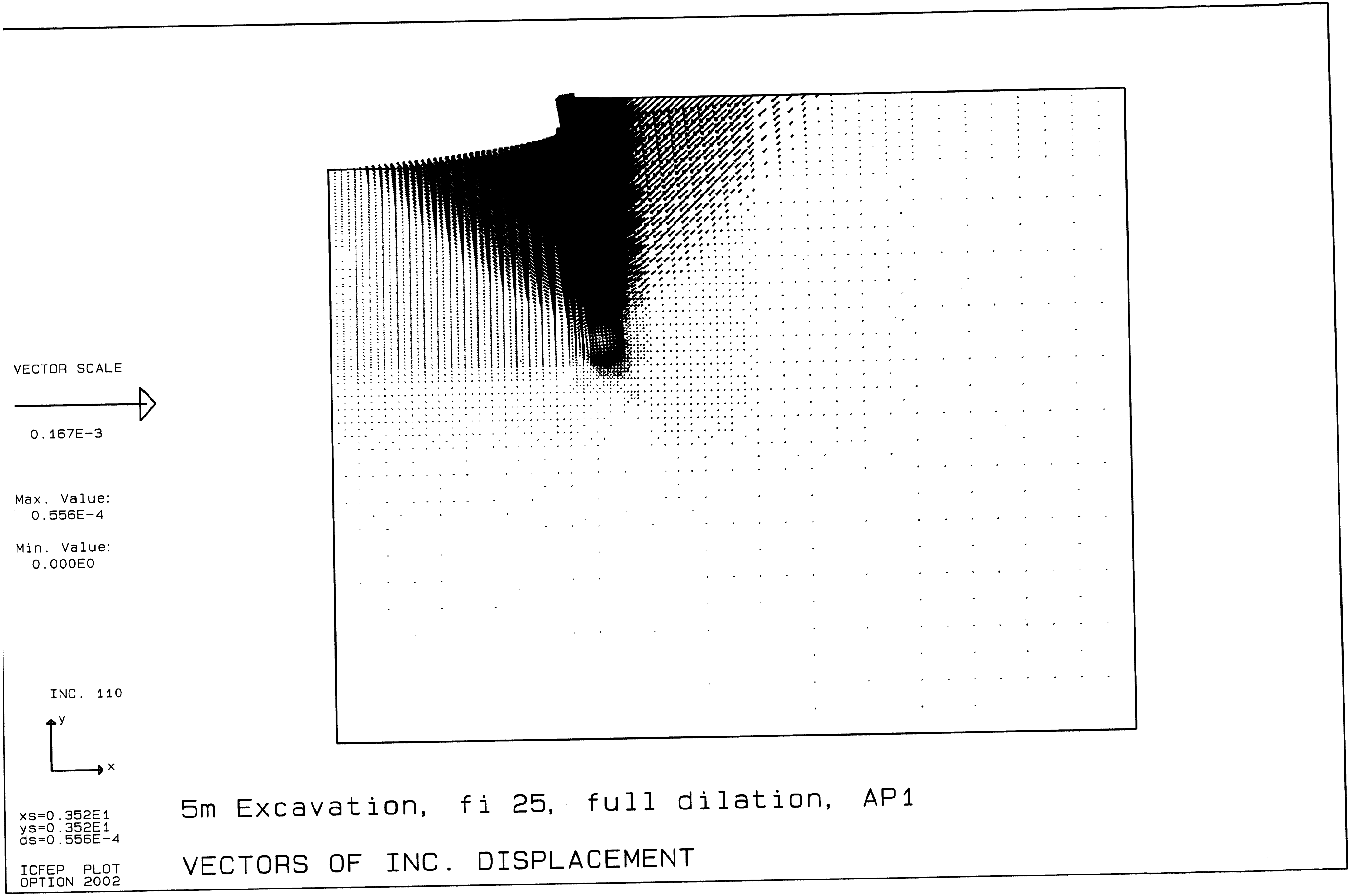
5m excavation, ϕ=25, v=0
F.O.S (-)
3
2.5
2
1.5
1
0.5
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δh (m)



5m Excavation, fi 25, zero dilation, AP1

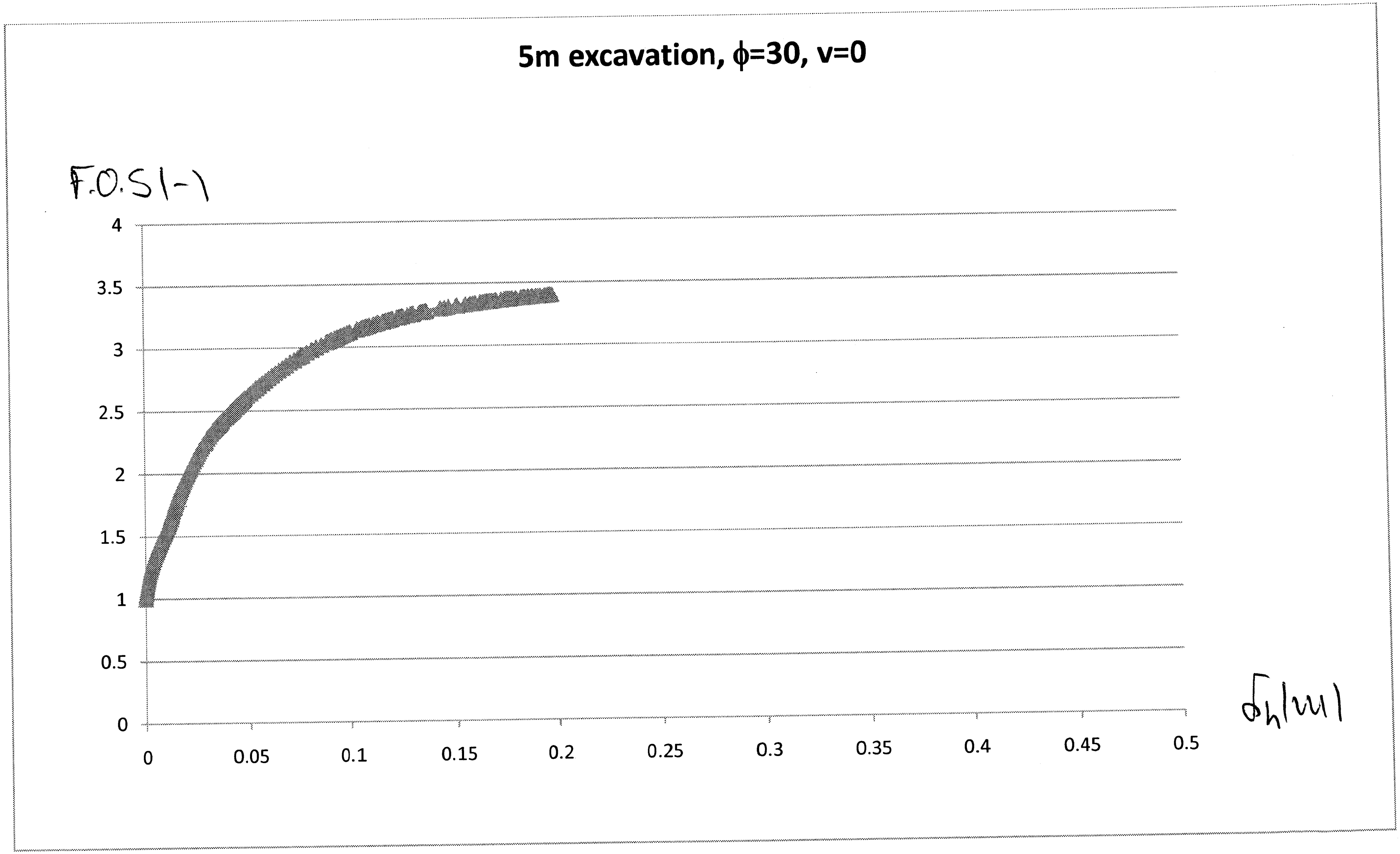
VECTORS OF INC. DISPLACEMENT

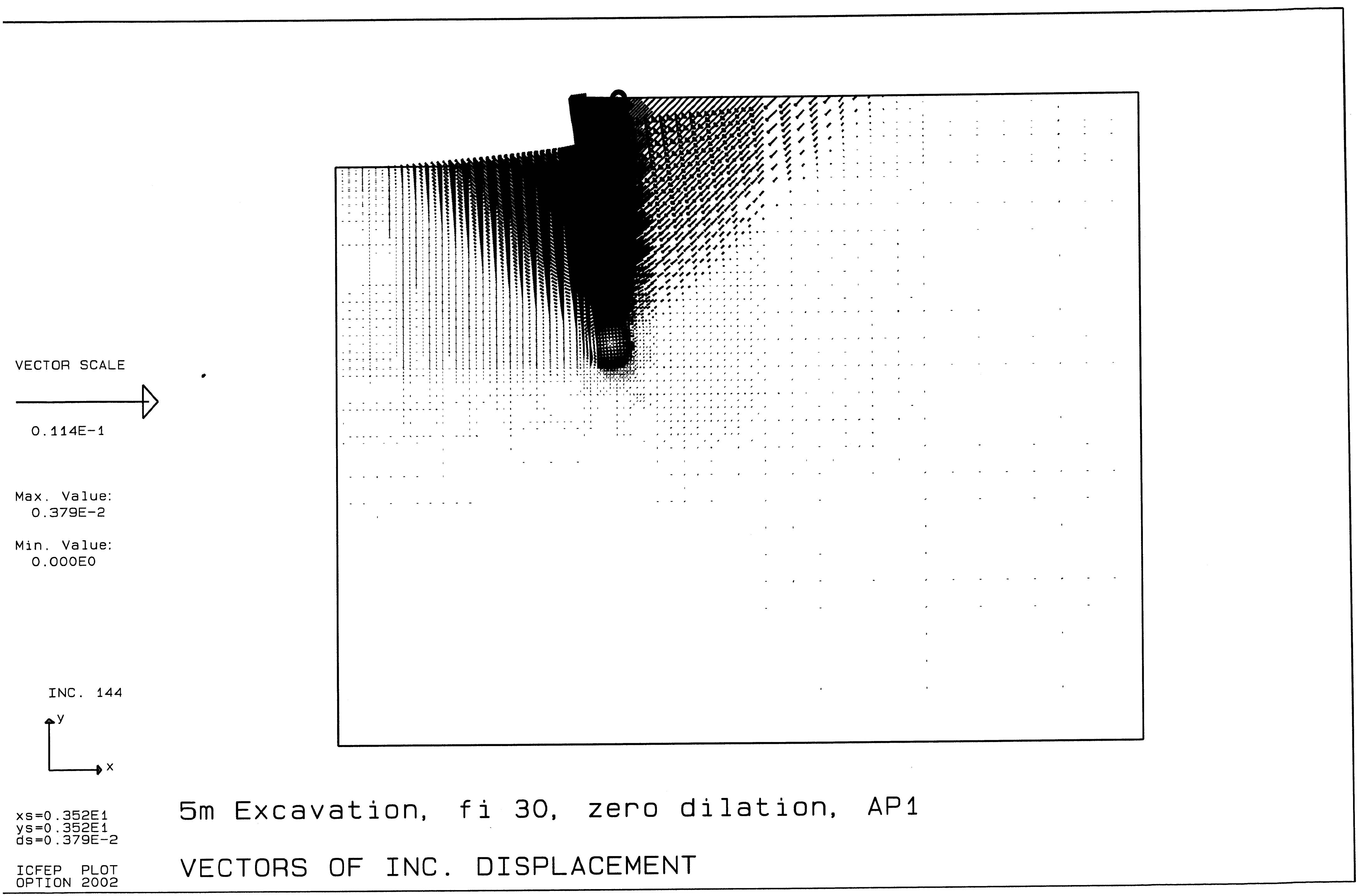

5m excavation, ϕ=25, v=25
F.O.S (-)
3
2.5
2
1.5
1
0.5
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δh (m)



5m Excavation, fi 25, full dilation, AP1

VECTORS OF INC. DISPLACEMENT


5m excavation, ϕ=30, v=0
F.O.S[-]
4
3.5
3
2.5
2
1.5
1
0.5
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δh[m]



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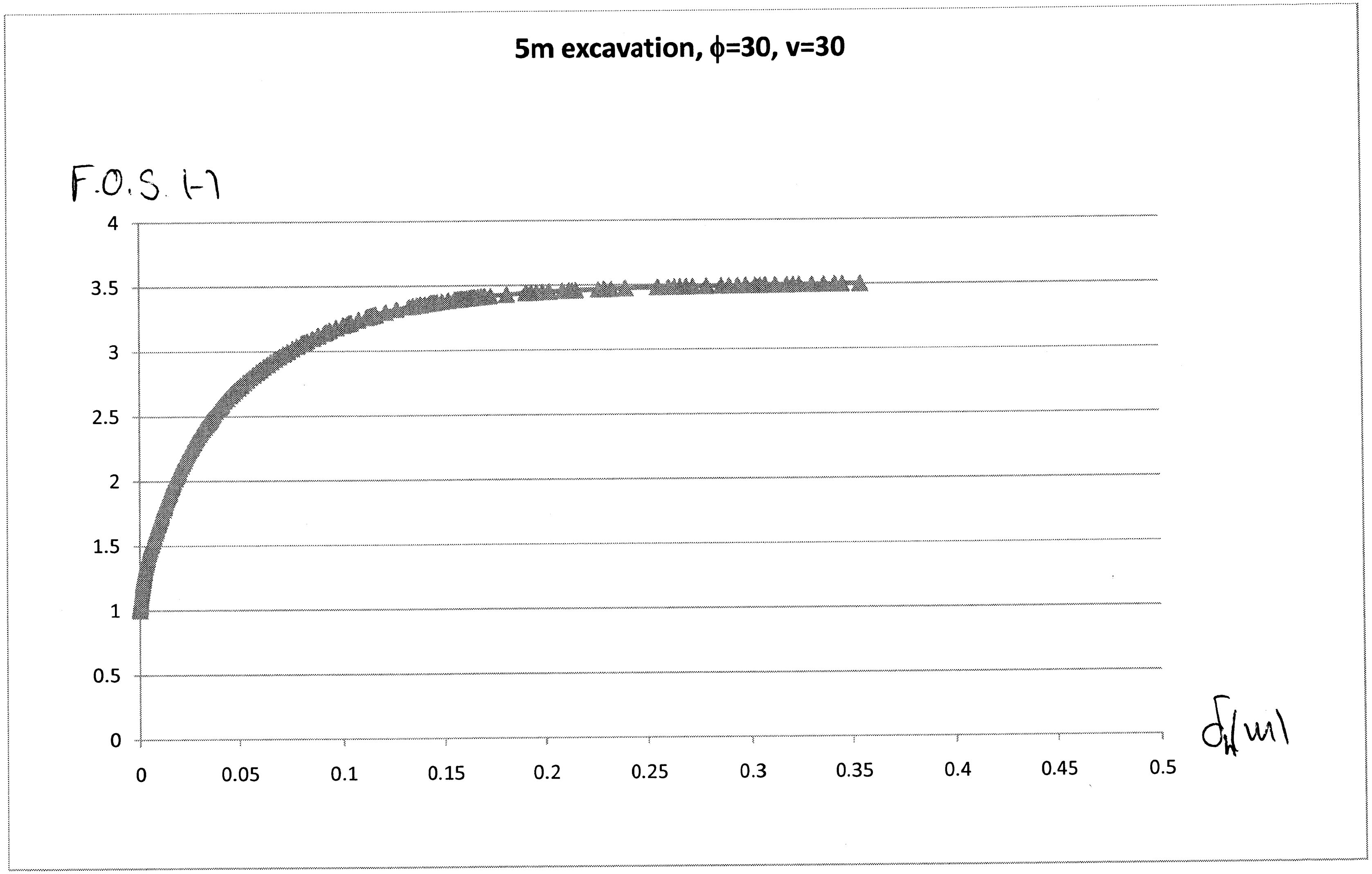
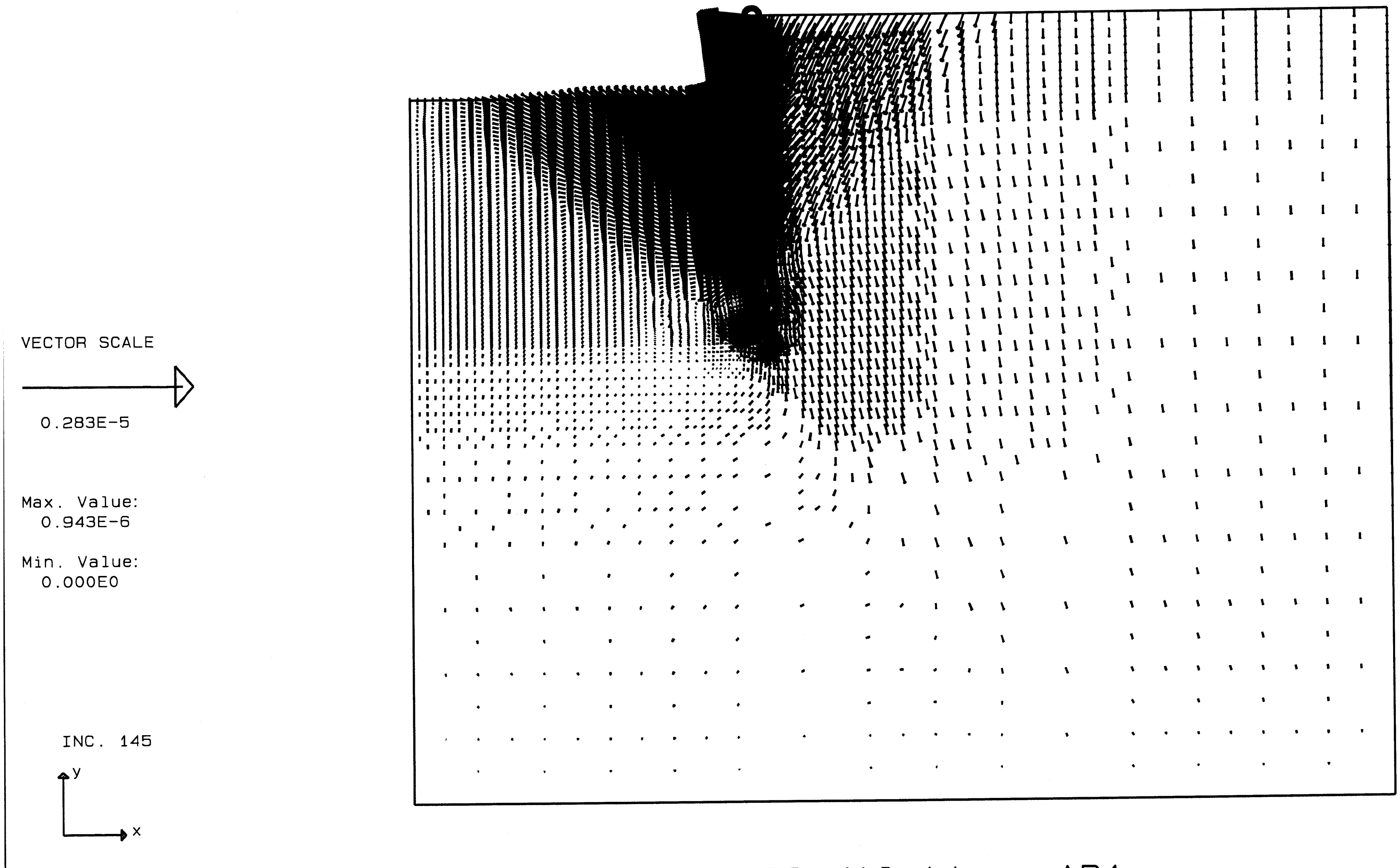
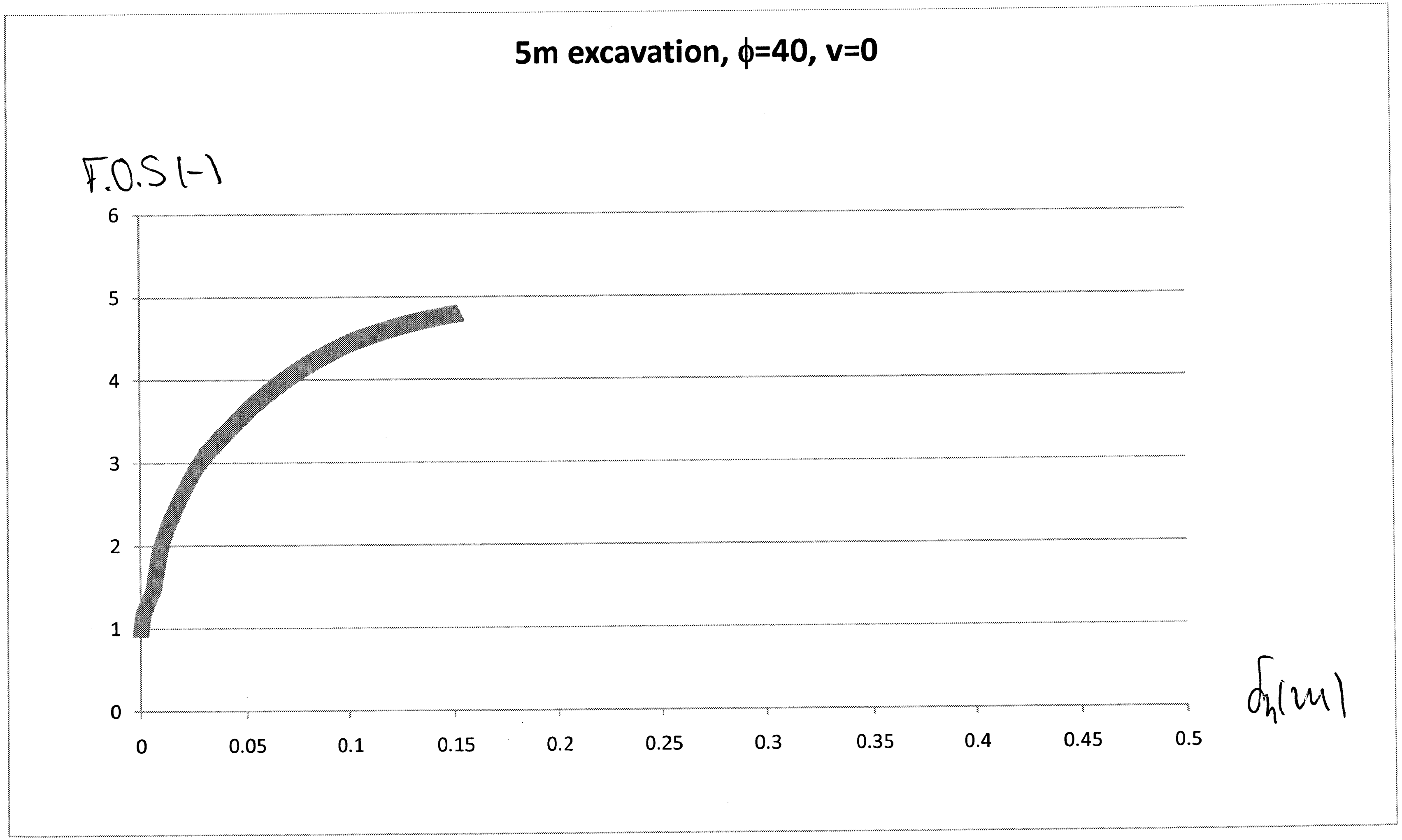
Max. Value:
0.379E-2
Min. Value:
0.000E0
INC. 144
y
x
xs=0.352E1
ys=0.352E1
ds=0.379E-2
ICFEP PLOT
OPTION 2002
5m Excavation, fi 30, zero dilation, AP1
VECTORS OF INC. DISPLACEMENT

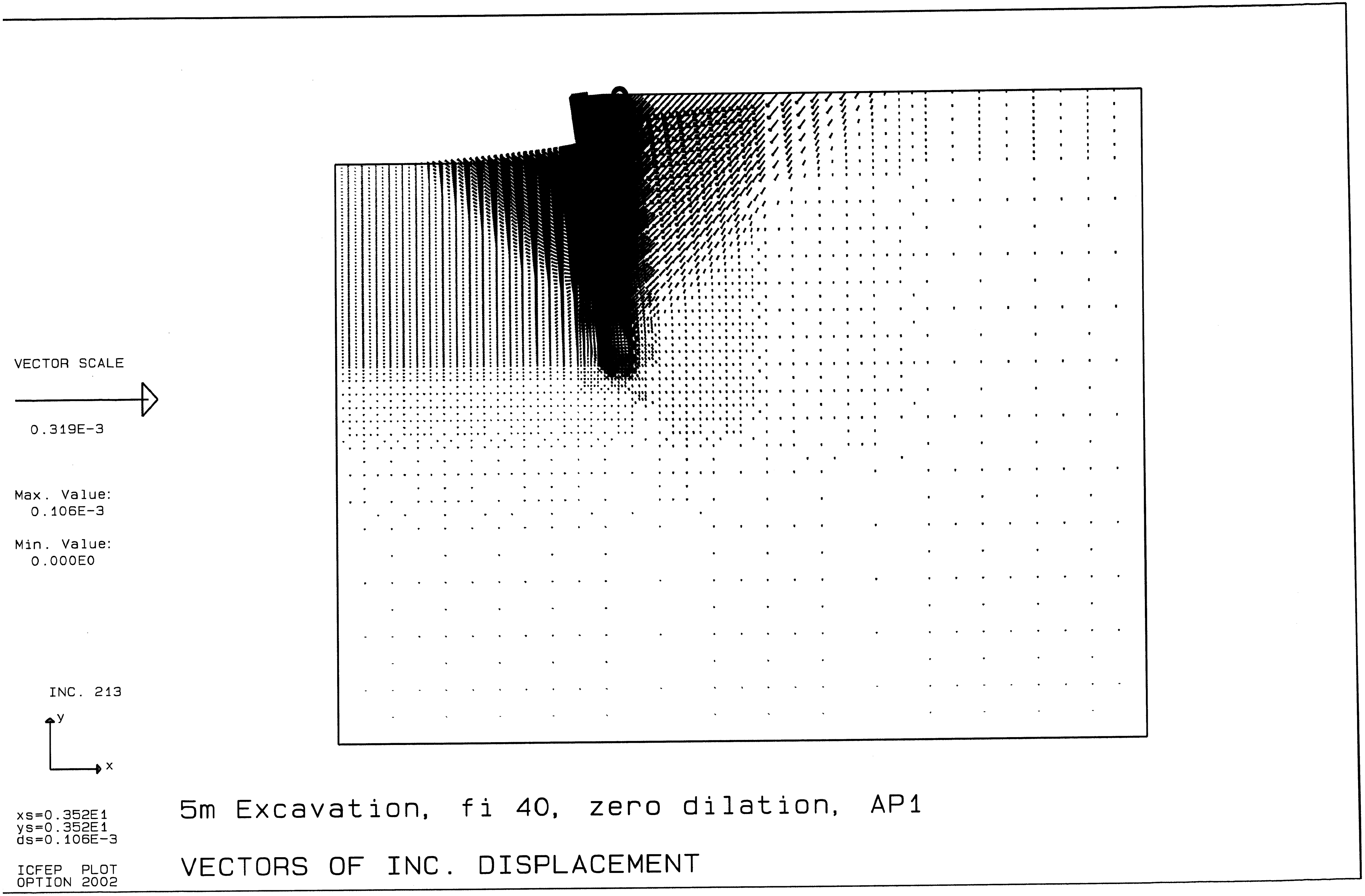

5m excavation, ϕ=30, v=30
F.O.S. (-)
4
3.5
3
2.5
2
1.5
1
0.5
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δH (m)



5m Excavation, fi 30, full dilation, AP1

VECTORS OF INC. DISPLACEMENT


5m excavation, ϕ=40, v=0
F.O.S (-)
6
5
4
3
2
1
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δh (m)



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ICFEP PLOT
OPTION 2002
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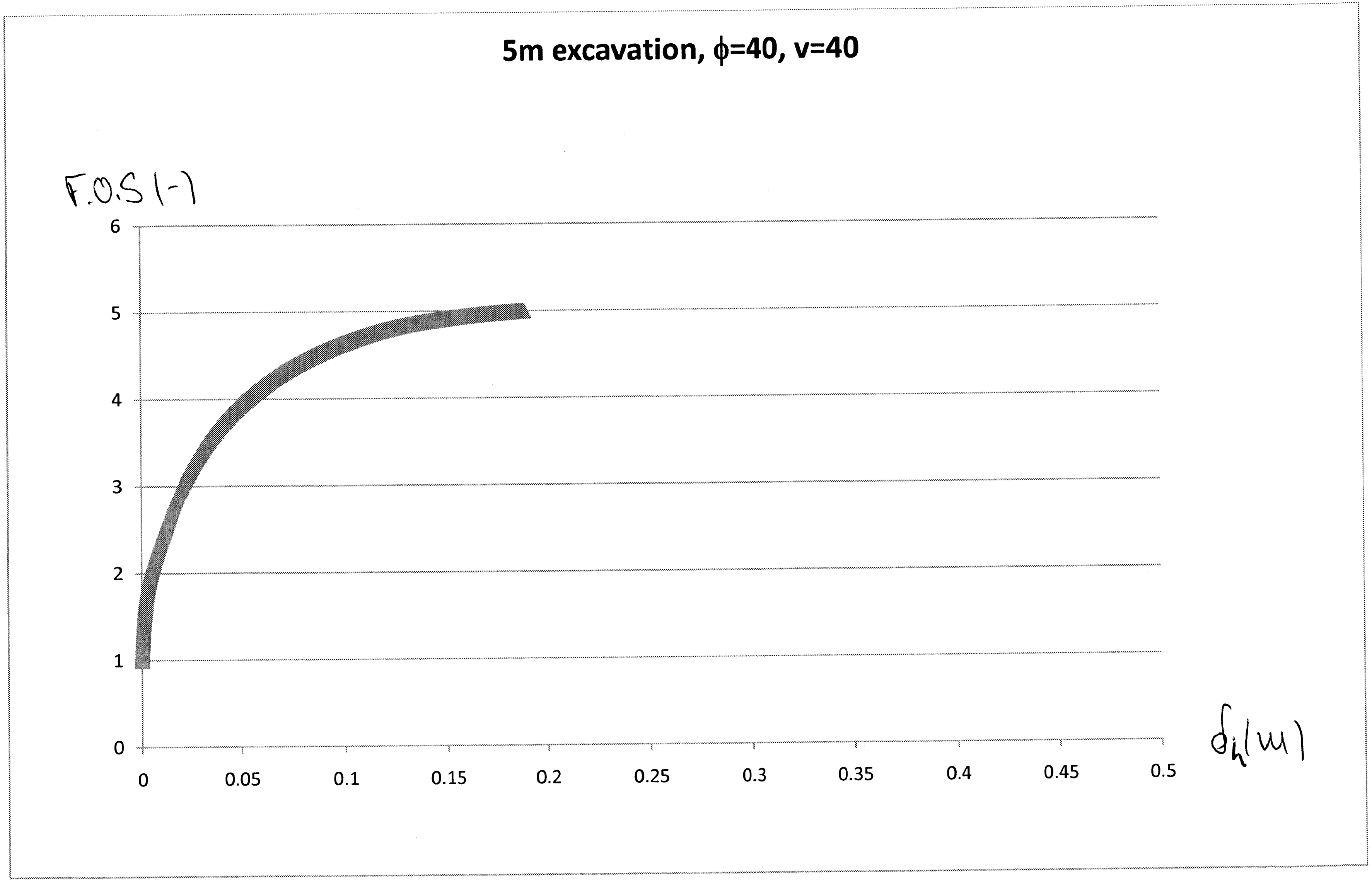
VECTORS OF INC. DISPLACEMENT


5m excavation, ϕ=40, v=40
F.O.S (-)
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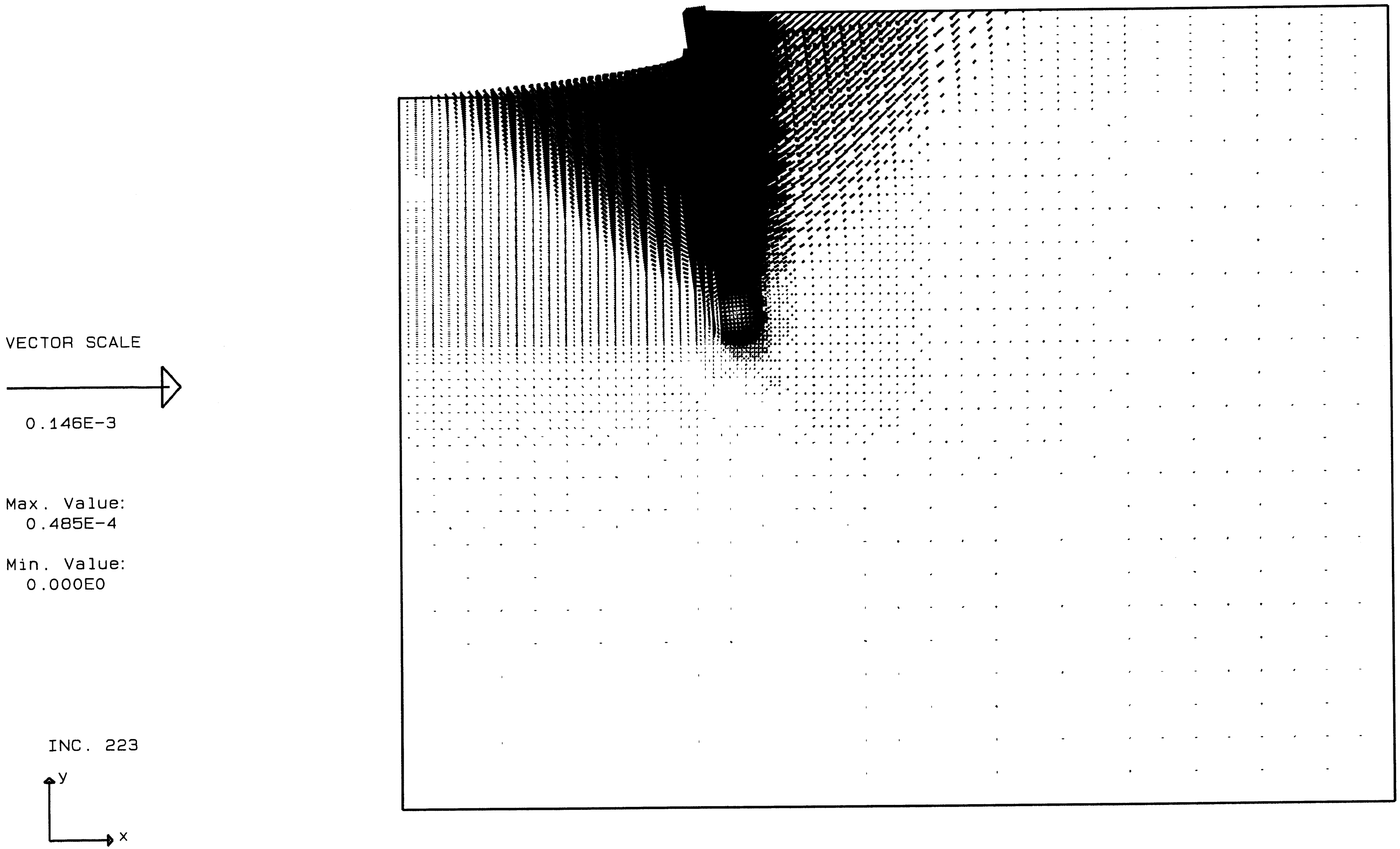
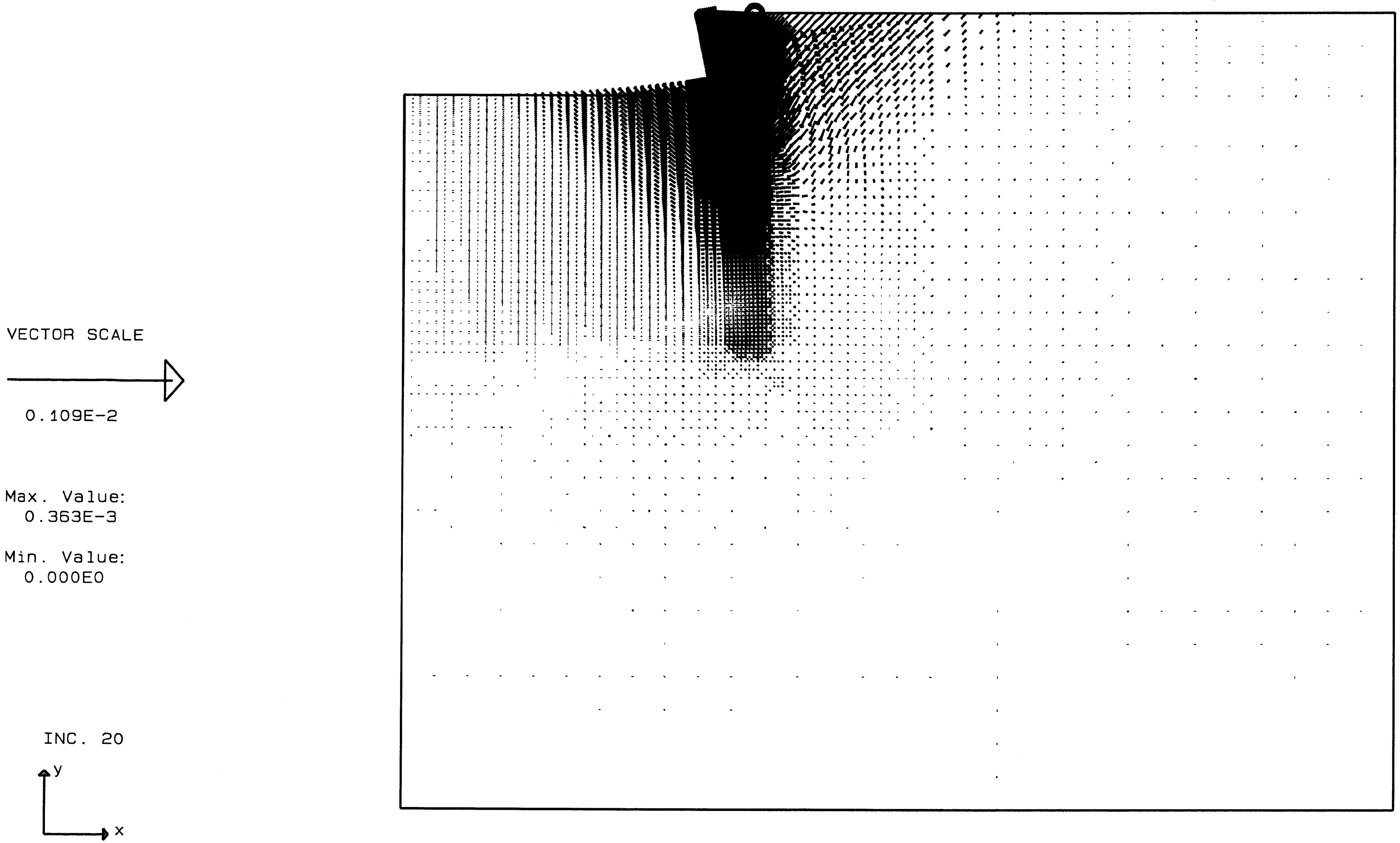
5
4
3
2
1
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δh (m)



5m Excavation, fi 40, full dilation, AP1

VECTORS OF INC. DISPLACEMENT



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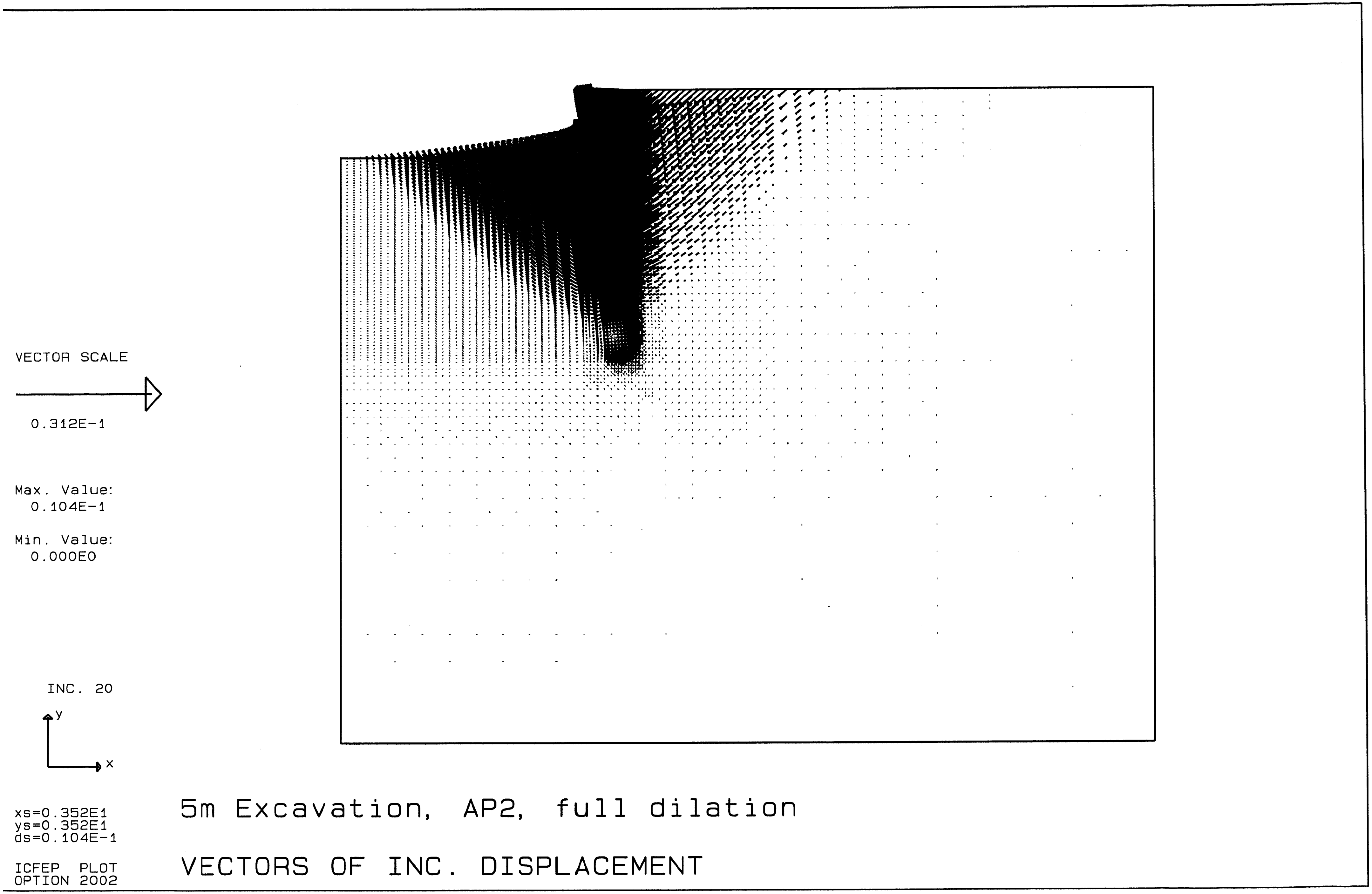
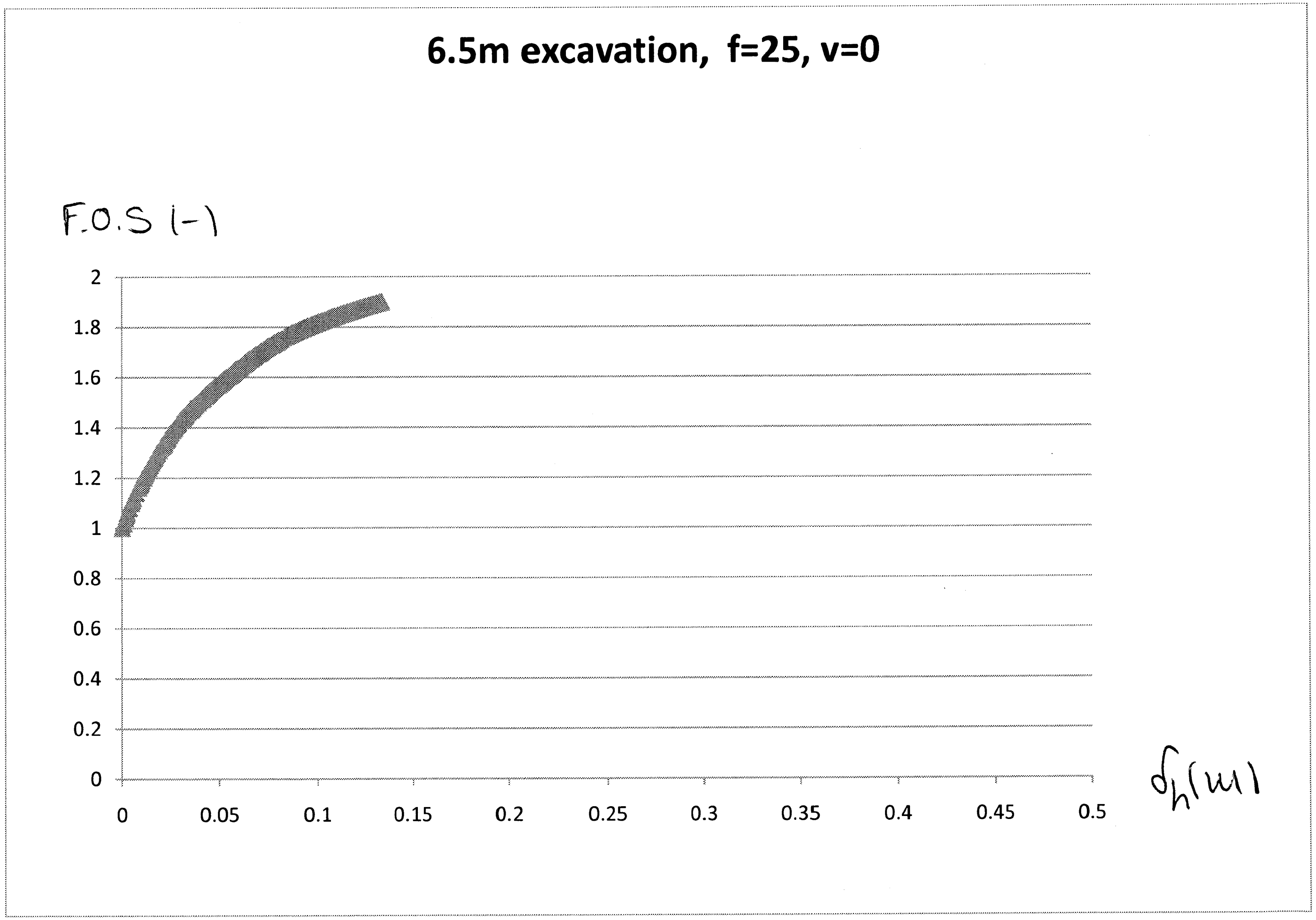
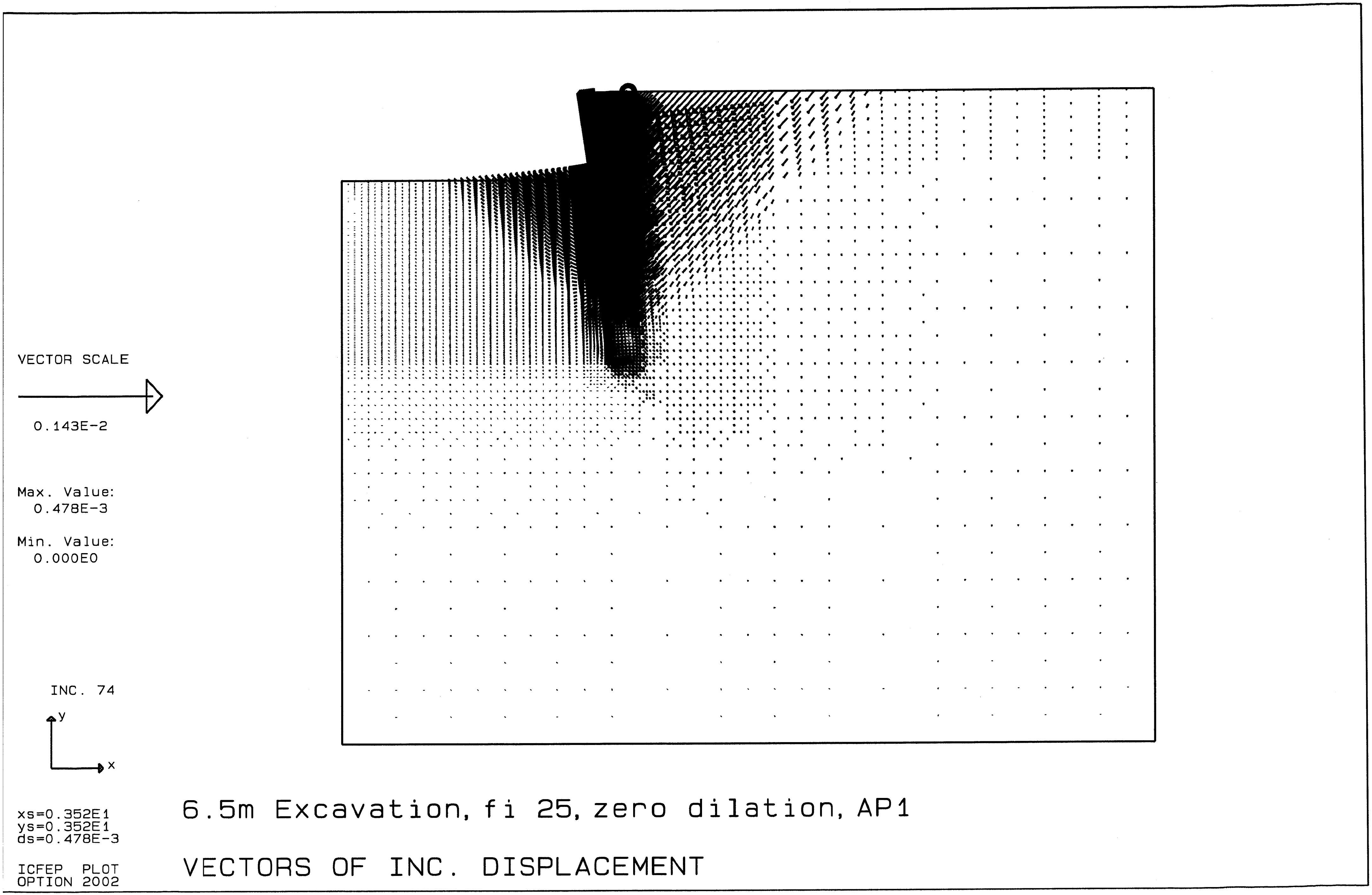
ICFEP PLOT
OPTION 2002

5m Excavation, AP2, zero dilation

VECTORS OF INC. DISPLACEMENT


VECTOR SCALE
0.312E-1
Max. Value:
0.104E-1
Min. Value:
0.000E0
INC. 20
y
x
xs=0.352E1
ys=0.352E1
ds=0.104E-1
ICFEP PLOT
OPTION 2002
5m Excavation, AP2, full dilation
VECTORS OF INC. DISPLACEMENT

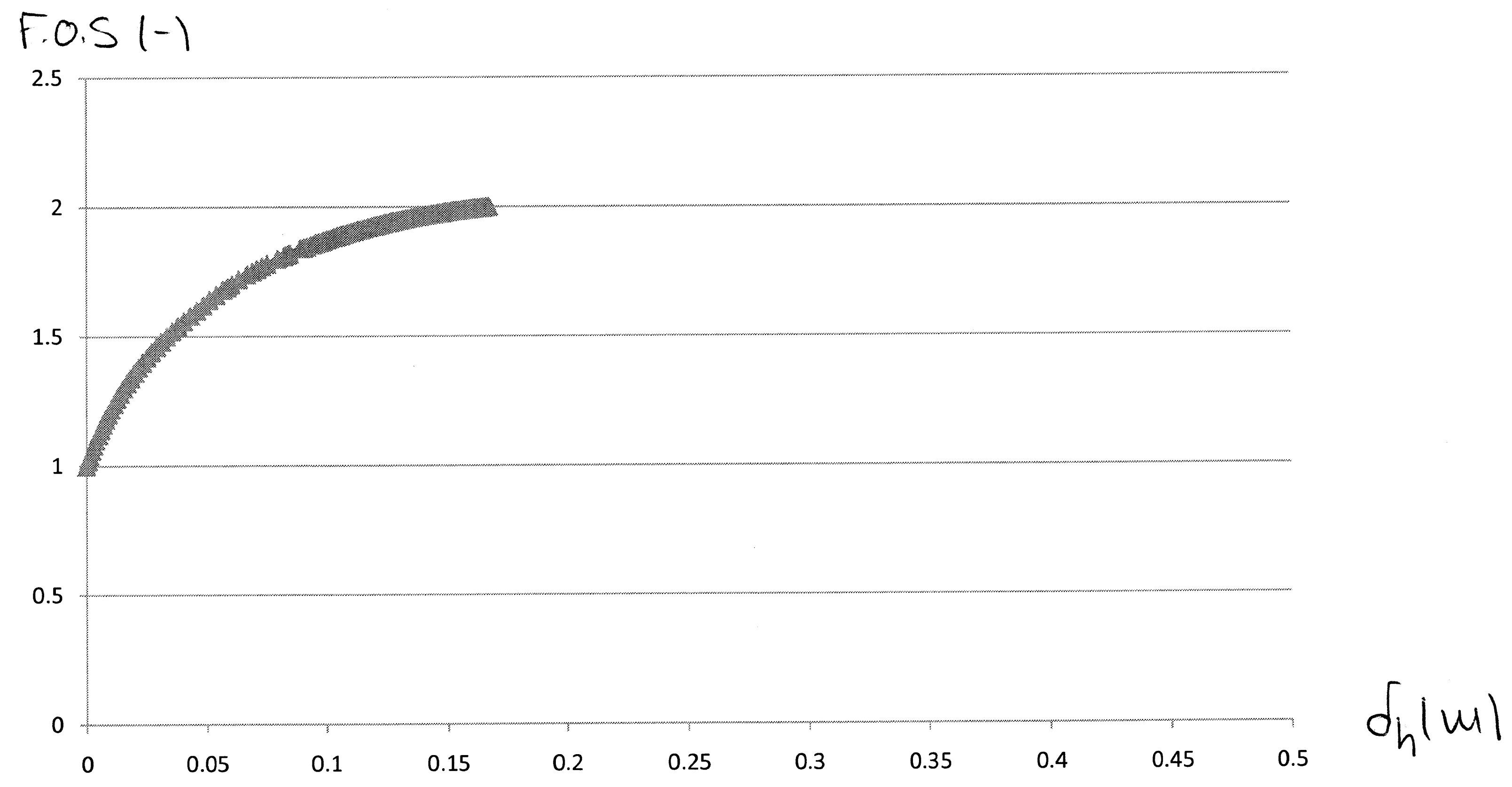

6.5m excavation, f=25, v=0
F.O.S (-)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δ_h(m)

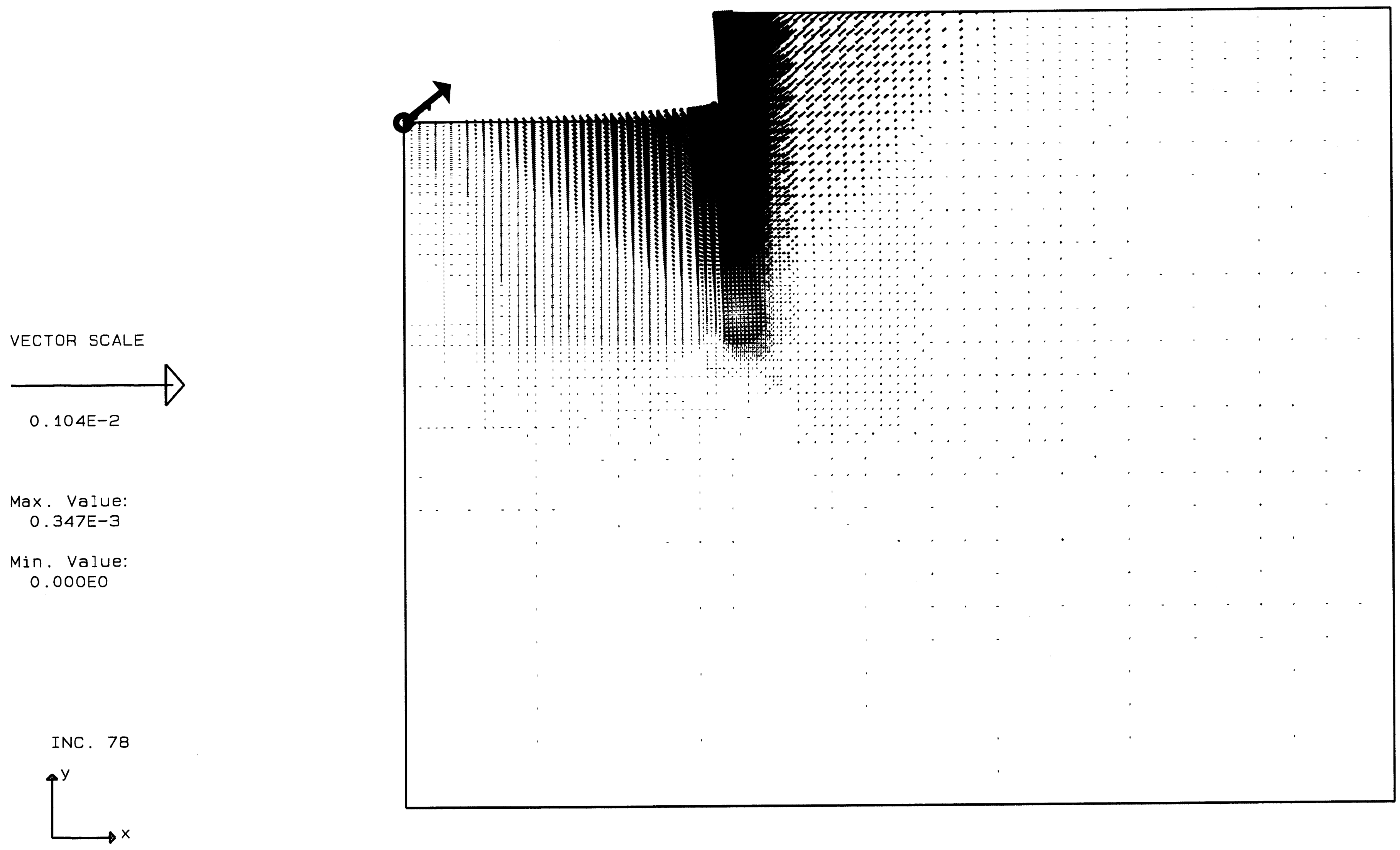


6.5m Excavation, fi 25, zero dilation, AP1

VECTORS OF INC. DISPLACEMENT

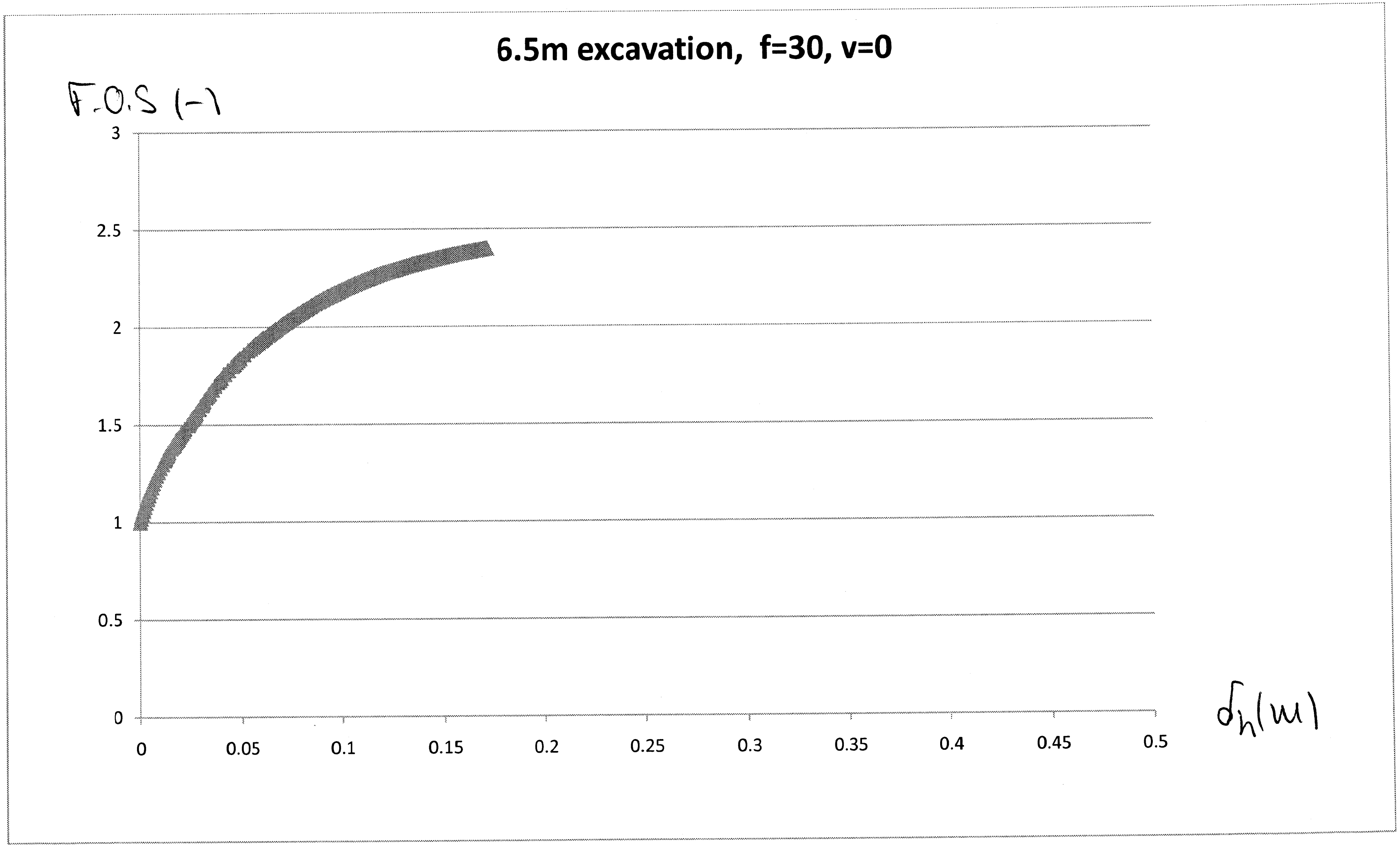
6.5m excavation, f=25, v=25



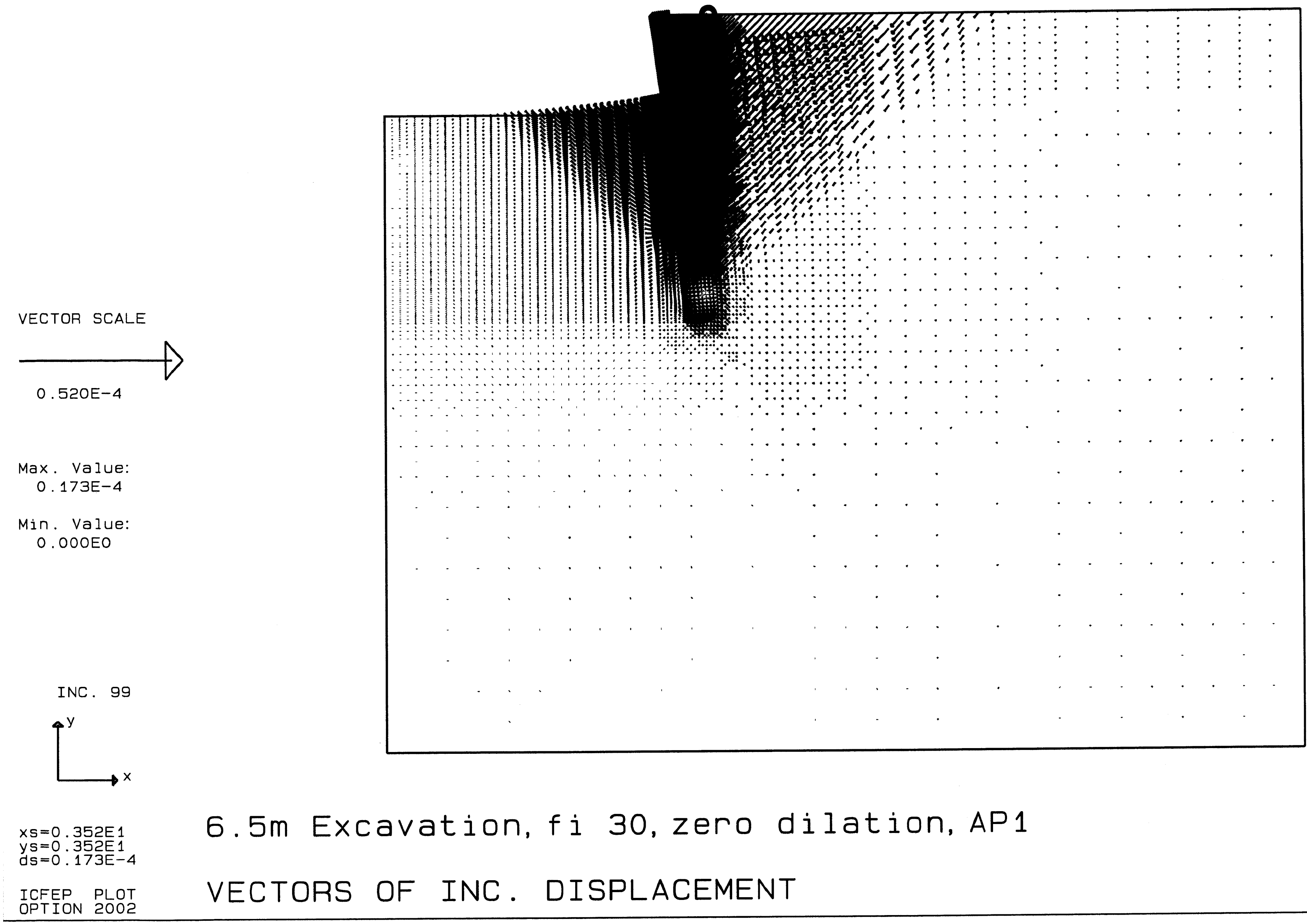


6.5m Excavation, fi 25, full dilation, AP1

VECTORS OF INC. DISPLACEMENT

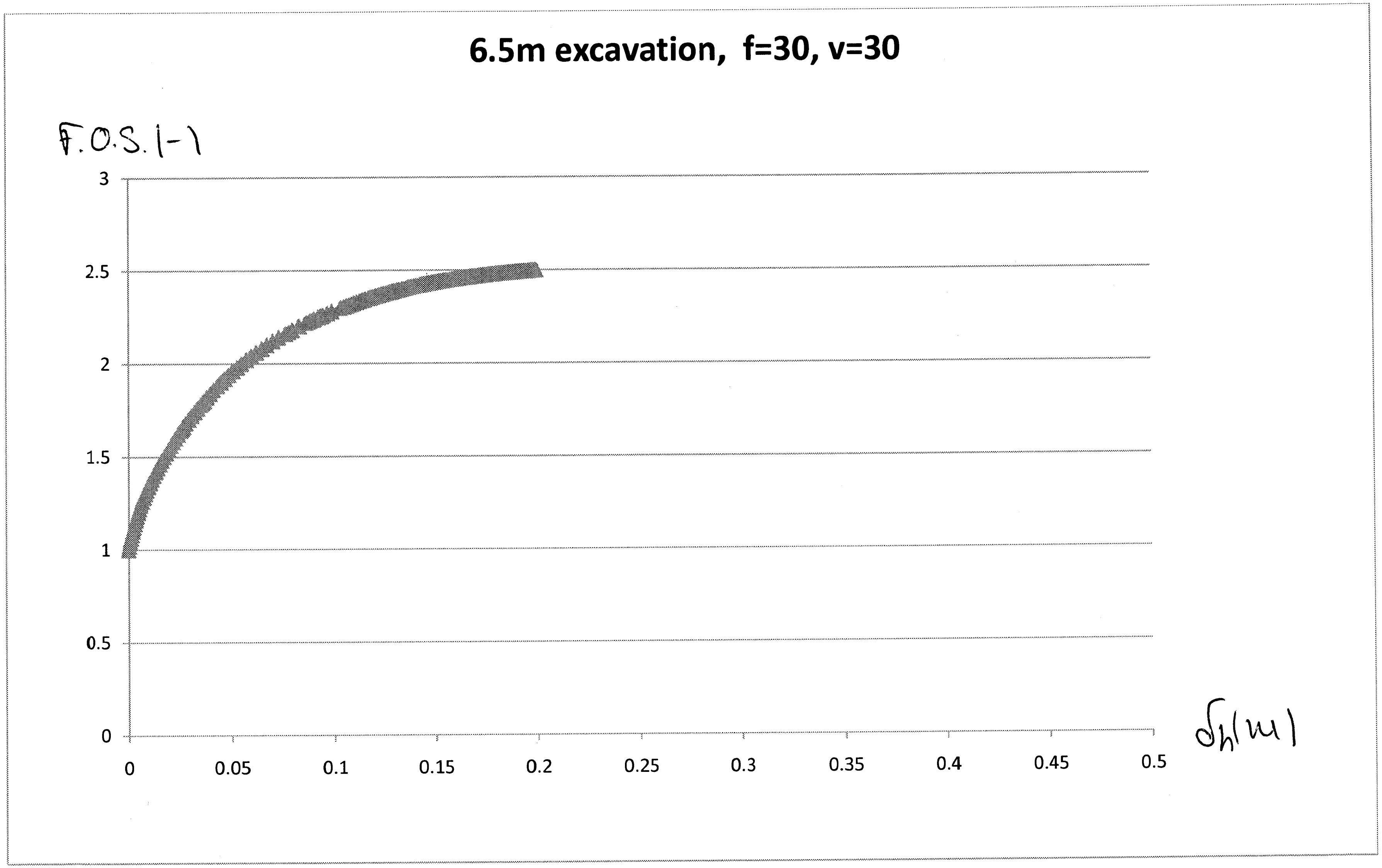


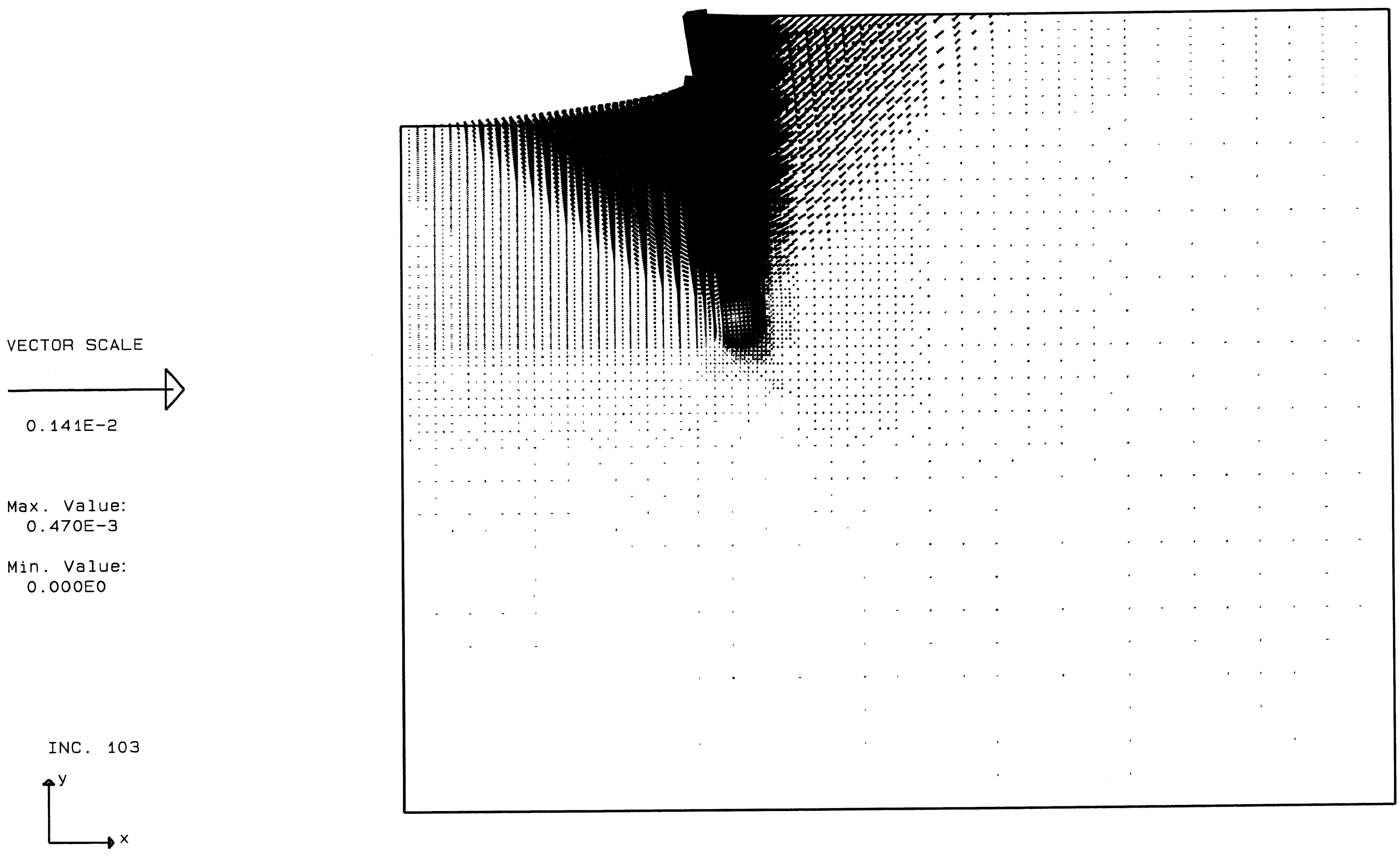
6.5m excavation, f=30, v=0
F.O.S (-)
3
2.5
2
1.5
1
0.5
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δh(m)



6.5m Excavation, fi 30, zero dilation, AP1

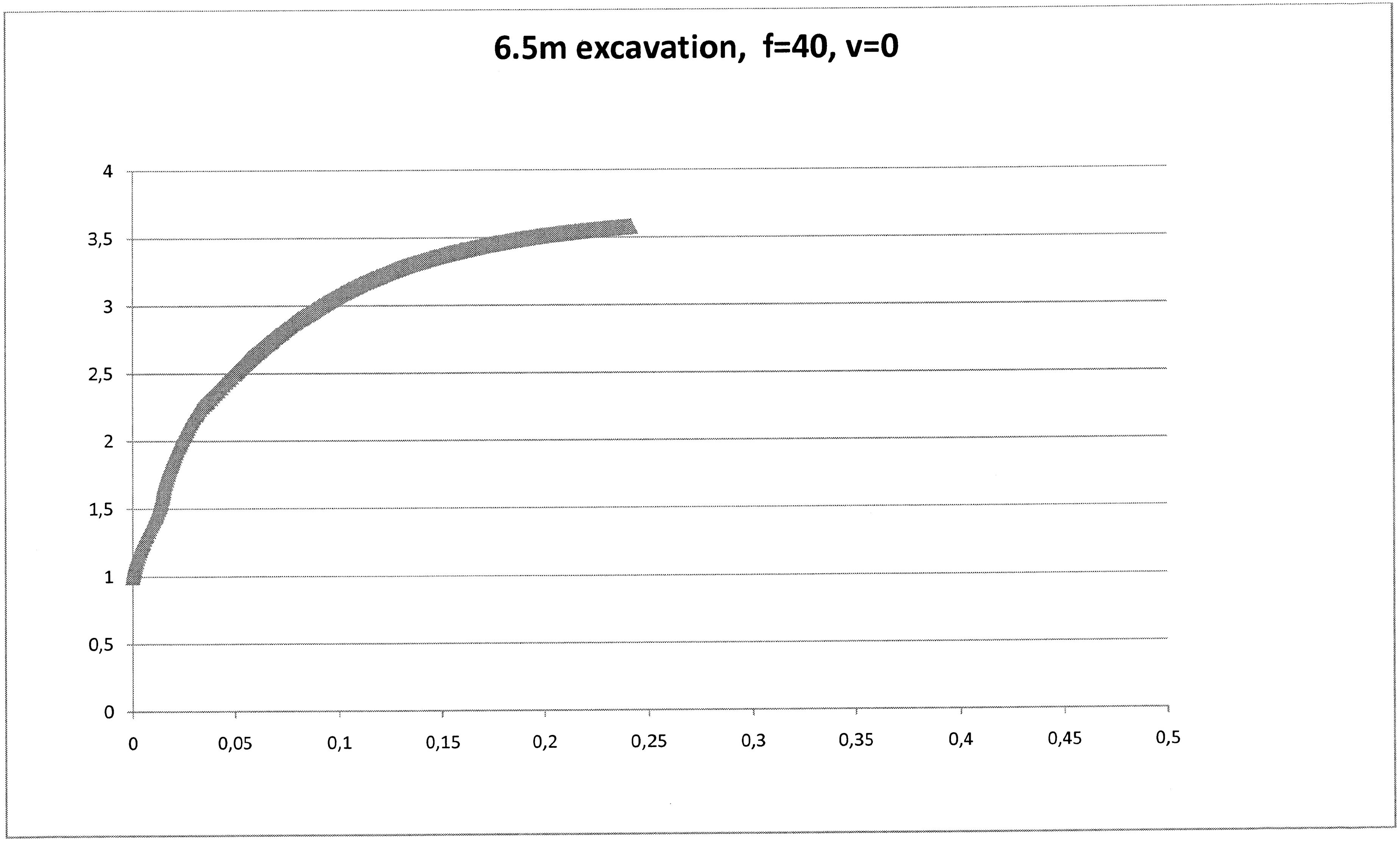
VECTORS OF INC. DISPLACEMENT

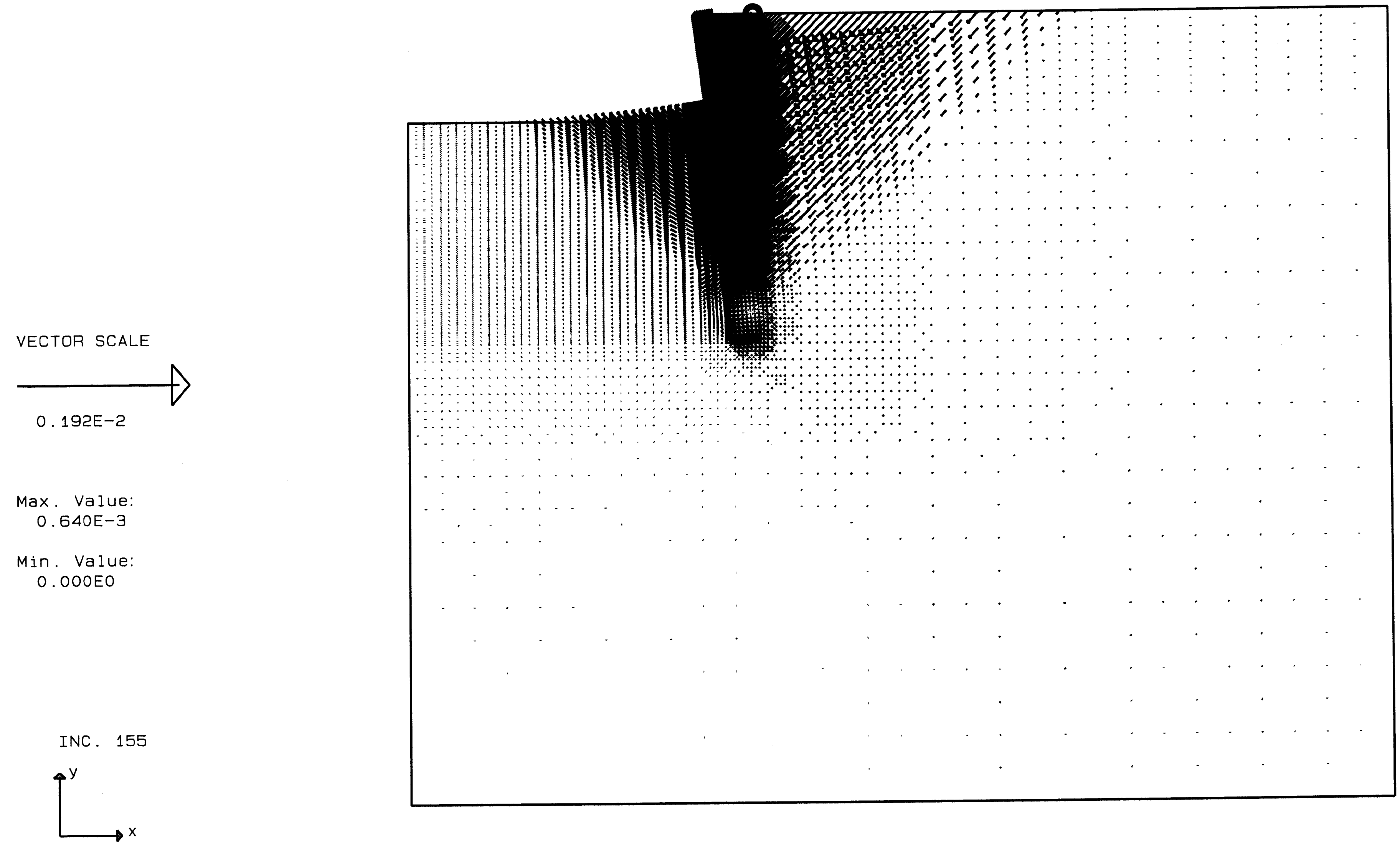

6.5m excavation, f=30, v=30
F.O.S. (-)
3
2.5
2
1.5
1
0.5
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
δh (m)



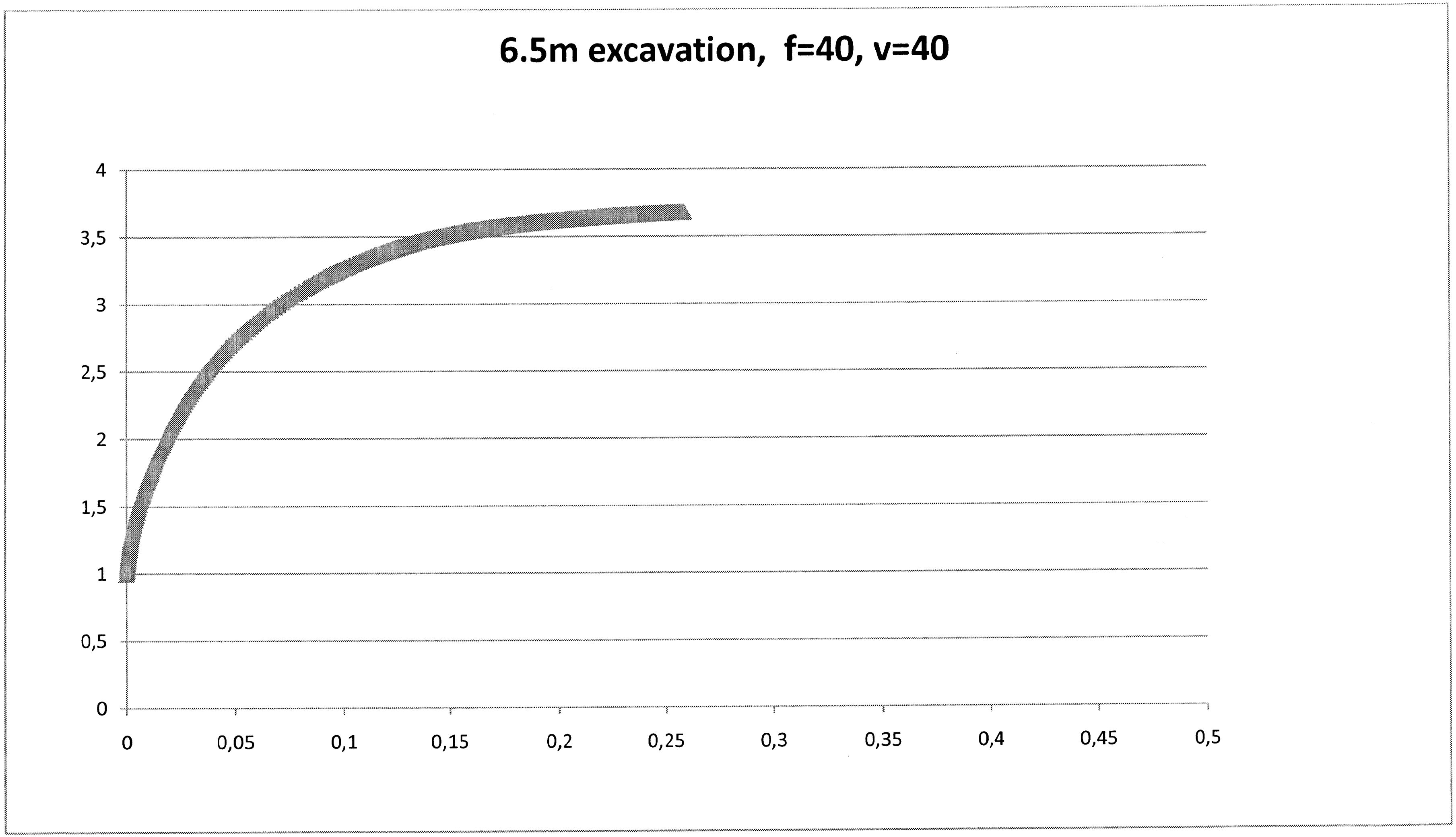
6.5m Excavation, fi 30, full dilation, AP1

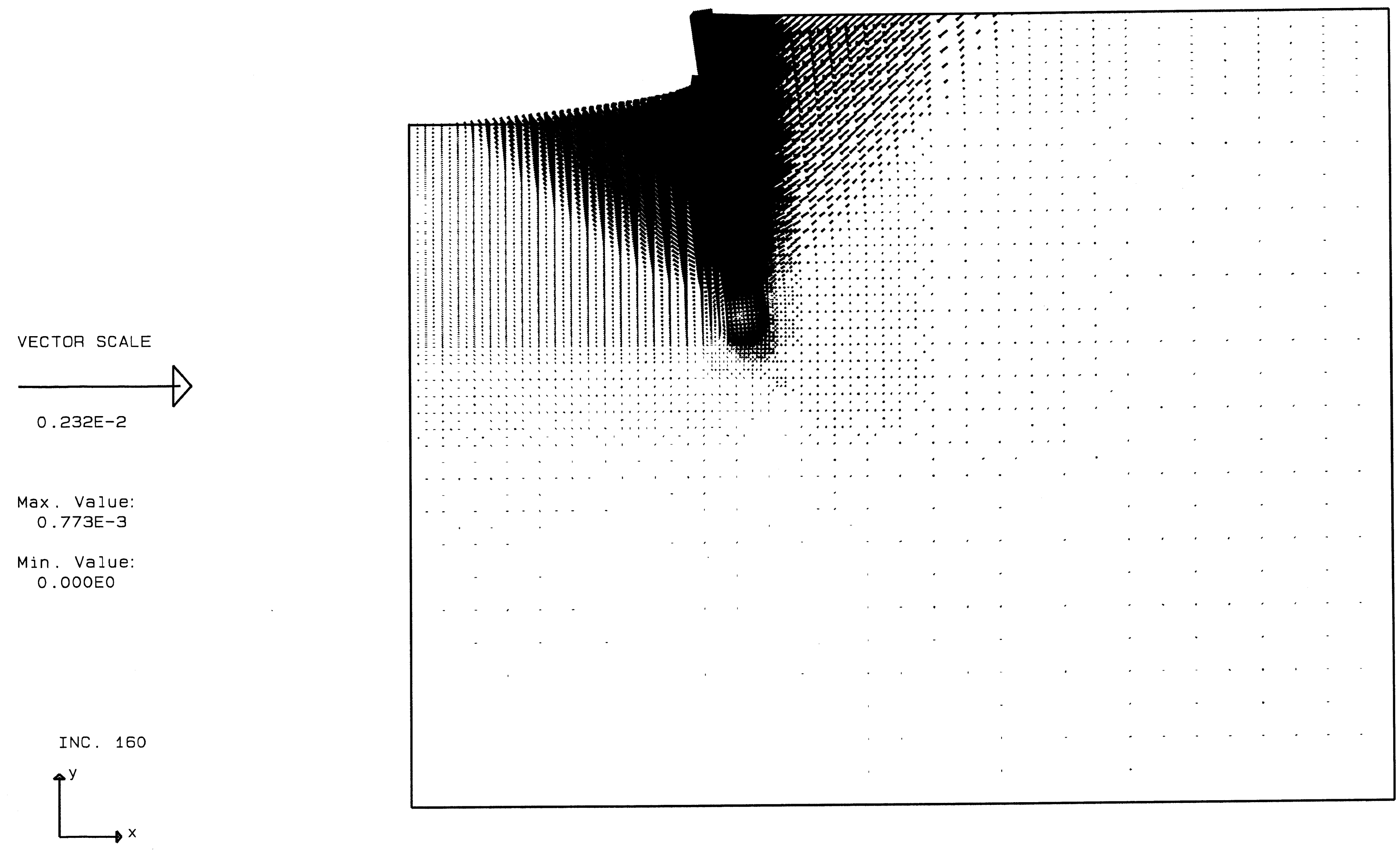
VECTORS OF INC. DISPLACEMENT


6.5m excavation, f=40, v=0
4
3,5
3
2,5
2
1,5
1
0,5
0
0
0,05
0,1
0,15
0,2
0,25
0,3
0,35
0,4
0,45
0,5



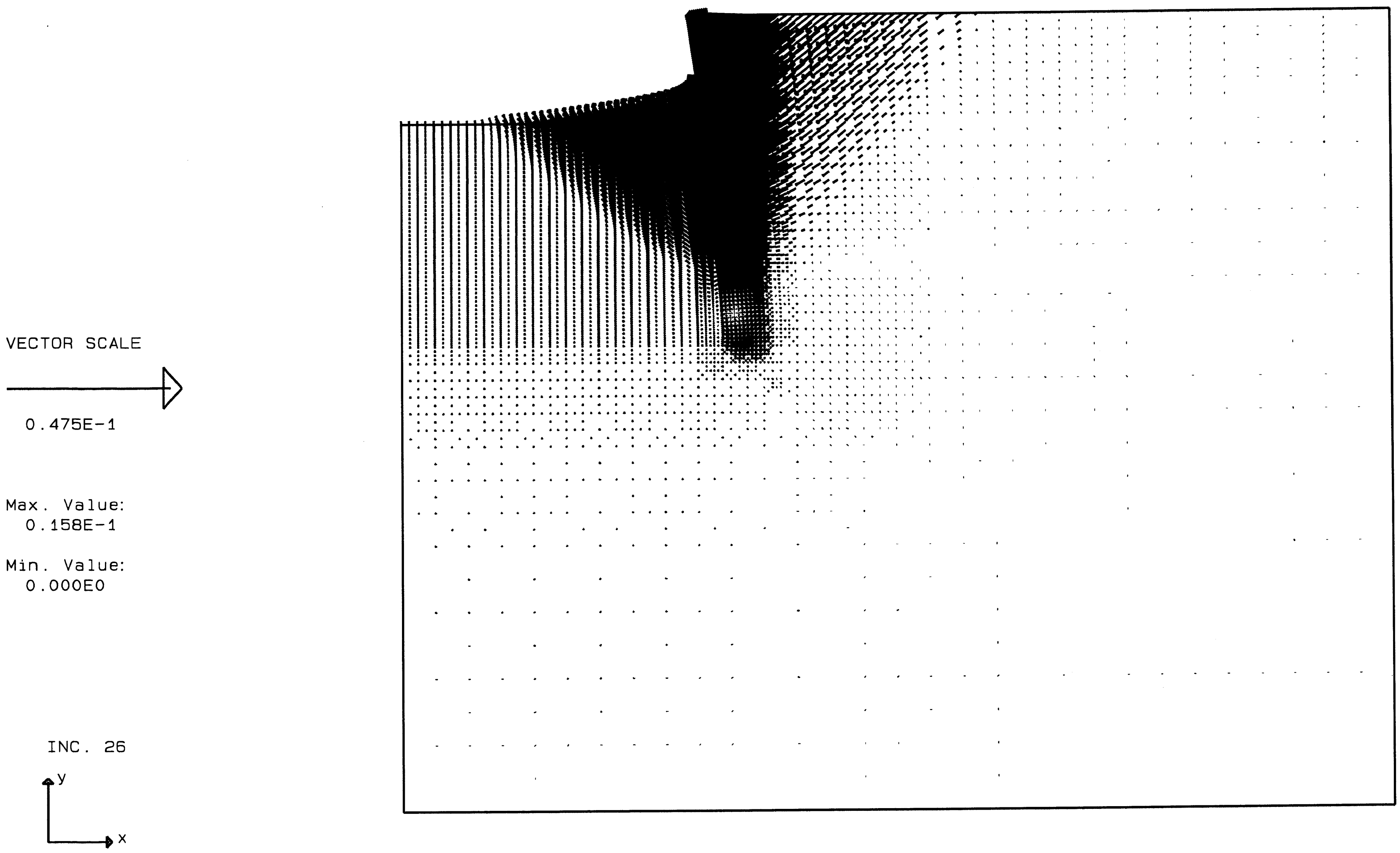
6.5m Excavation, fi 40, zero dilation,
AP1
VECTORS OF INC. DISPLACEMENT


6.5m excavation, f=40, v=40
4
3,5
3
2,5
2
1,5
1
0,5
0
0
0,05
0,1
0,15
0,2
0,25
0,3
0,35
0,4
0,45
0,5



6.5m Excavation, fi 40, full dilation,
AP1

VECTORS OF INC. DISPLACEMENT



6.5m Excavation, AP2, full dilation

VECTORS OF INC. DISPLACEMENT

Appendix C

Explained example of code implemented.

Incremental Analysis

```
*PROGRAM CONTROL
START   68, 1      Increment at start, phase.
STOP   500, 1          "      " stop,  "
*comm INDISC /home5/msc6/DATA/NACHO/MESH/MESH.SVE  if Increment at 1
INITIAL_STRESS /home5/msc6/DATA/NACHO/MESH/ins/i25d  domain info
INDISC RUN7.SVE → info until increment of start
NEWDATA overwrite in
HPPLOTTER Visualization Tool
AUTOMATIC_INCREMENTATION Initiates substepping stress point algorithm
NONSYMMETRIC Allow to solve with nonsym matrix (important storage memory)
SAVE
PLANE 1
NONLINEAR Nonlinear solver using incremental algorithm
ITERATION 1, 500  control the number of iterations per increment
ALPHA     1, 500, 5  α - acceleration technique in the non-linear solver
FLOW 10000, 1000, 2000 parameters which control the flow sub-stepping
                       in elastoplastic analysis.
*ENDCONTROL
*DEFINE
*comm ---------------------------------
*UPDATE STIFFNESS  From 1 to 1000, the parameter applies to every stage.
*comm -------------------------------- → preconditioning parameter
1:   1, 1000, 1, 300, 1, 1
*comm ---------------------------------
*LINEAR MATERIAL PROPERTIES   defined for elastic states
*comm ---------------------------------
*comm   Soil                             dry density
1:  1.0, 20.0, 1.0, 1.0E5, 0.2,  0.0     Young modulus
*comm   Wall                             Poisson ration
2:  1.0, 24.0, 1.0, 30.0E6, 0.15, 0.0
*comm ---------------------------------
*NONLINEAR MATERIAL PROPERTIES  Constitutive models
*comm ---------------------------------
*comm   Soil        φ     ψ
1: 16.0, 0.0, 25.0, 0.0, 0.1   Mohr-Coulomb φ=25 ψ=0
*comm   Wall                   no hardening/softening
2:  1.0 → Linear elastic
*comm ---------------------------------
*PRESCRIBED DISPLACEMENTS
*comm ---------------------------------
1:   1, 1000, 2,     1,     3, 0 → From increment 1 to 1000
2:   1,    4, 1, -9726, -1290, 0   displacements in direction 2 (vertical)
3:   5,    8, 1, -9510, -1290, 0   in element(s) n°3 set to 0 (bottom)
4:   9,   12, 1, -9294, -1290, 0
5:  13,   16, 1, -9078, -1290, 0 → to allow for excavation (variation over the increments)
6:  17, 20, 1, -8862, -1290, 0     From increment 9 to 12
7: 21, 24, 1,-8646, -1290, 0       displacements in direction 1
8: 25, 26, 1,-8538,-1290, 0        (horizontal) from point 9294 to
9: 27, 1000, 1, -8322,-1290,0      point 1290 set to zero. (boundary)
*comm ---------------------------------
*CHANGE MATERIAL PROPERTIES
*comm ---------------------------------
*comm   Soil to wall
1:  1, 2, 1     Change section 1 to material properties tag 2
*comm ---------------------------------
*EXCAVATION     27 increment 27
*comm ---------------------------------
*comm   Excavation to 5m depth
1:   1,  4,  2
2:   5,  8,  3
3:   9, 12,  4   → increments 9 to 12 excavate
4:  13, 16,  5     section 4.
5:  17, 20,  6
6: 21, 24, 7
7: 25, 26, 8
```

```
*comm ----------------------------------------
*INCREASE FOS        From increment 27 to 100 increase
*comm ----------------------------------------
1:  27, 100, 1, 7   FOS according to fac 1 of Section 7
*comm ----------------------------------------
*FOS CHANGE
*comm ----------------------------------------
1:  2.0, 0.02      Increase of 0.02 per increment  (in 2 increments
                                                     minimum?)
*comm ----------------------------------------
*SECTIONING
*comm ----------------------------------------
*comm   Wall
1:  1, 131         Section 1 assigned to element (1) 131
*commm Excavation to 5m
2:  2, 1766,-1785, 3158,-3229
3:  2, 1746,-1765, 3086,-3157
4:  2, 1726,-1745, 3014,-3085   Section 4 defined by points (2)
5:  2, 1706,-1725, 2942,-3013        from ... to ...
6:  2, 1686,-1705, 2870,-2941
*comm   FOS        Section 7 assigned to material (3) n° 1
7:  3, 1
*comm ----------------------------------------
*PLOT
*comm ----------------------------------------
1:  0,   1, 1, 1010
2:  4,   20, 4, 1002  -> From increment 4 to 20, every 4 increments
3:  4,   20, 4, 1004     plot graph 1002
4:  4,   20, 4, 2012
5: 20,  70, 5, 2002      2002: "mechanism plot"
6: 71, 100, 1, 2002
*comm ----------------------------------------
*XY-GRAPH   For increment 20 to 70 every 5 represent accumulated increment F.O.S (19403)
            at point 1036 in ordinates against incremental horiz. displacement (14051) in
*comm ----------------------------------------   element 253.
1:  20, 70, 5, 19403, 1, 1036, 1,  14051, 3, 253,0,  0,0,0,1,1
2:  20, 70, 5, 19403, 1,  996, 1,  14051, 3, 253,0,  0,0,0,1,1
3:  71, 100, 1, 19403, 1, 1036, 1,  14051, 3, 253,0,  0,0,0,1,1
4:  71, 100, 1, 19403, 1,  996, 1,  14051, 3, 253,0,  0,0,0,1,1
*comm ----------------------------------------
*SAVE   Save state from increment 27 to 70 every 5 increments
*comm ----------------------------------------
1:   4,  26, 4
2:  27,  70, 5
3:  71, 200,1
*comm ----------------------------------------
*ENDDATA
*comm ----------------------------------------
```