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Elastic and creep settlements of rock fills

Les tassements elastiques et secondaires dans les remblais d'enrochement

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ABSTRACT

Rock fills are widely used to ensure a proper foundation ground for large industrial plants, airports and other structures where very strict tolerances are required for the magnitude of settlements, differential settlements and rotations. With these very strict requirements (allowable settlements within few centimeters) for the whole life time of the structures, the estimation of elastic and particularly of creep settlements of the rock fills becomes a crucial issue in the foundation design. The few settlement measurements on rock fill embankments and dams, available in literature, were used to develop a simplified method of creep and elastic settlement calculation and to estimate preliminary settlement parameters for foundation design of Troll Onshore Plant at Kollsnes (As Norske Shell), Snøhvit LNG Plant at Hammerfest (STATOIL) and Ormen Lange Onshore Terminal at Aukra (Norsk Hydro), in Norway. Due to uncertainties in creep parameters, settlement measurements has been performed at Kollsnes rock fill over a period of 11 years. These are used to verify the assumptions and parameters of the proposed method. The comparison between predicted and measured settlements shows a fairly good agreement for the creep settlements of the fill under own weight.

RÉSUMÉ

Les remblais sont largement utiliseé pour les infrastructures industrielle et de transport. Les tolerance severes qui sont imposeés pour les tassements des fondations (limitation de tassements totales et differentielles a des valeurs très faible) même a la fin de la periode d'exploitation de structures fait que l'estimation des tassements elastiques et secondaires est devenue un probleme imporatnte pour le dimensionnement de foundations. Les tassements mesureés aux quelques ouvrages des remblais et de barrages en materiaux rocheuses ont étaites utiliseès pour developer une methode simple de calcul des tassements secondaires et de calibrer les parametres. La methode a etè utiliseè pour dimensionnement des fondations pour les Combinats Petrochimiques de Troll a Kollsnes (AS Norske Shell), de Snøhvit, a Hammerfest (STATOIL) et de Ormen Lange a Aukra (Norsk Hydro), tous en Norvège.

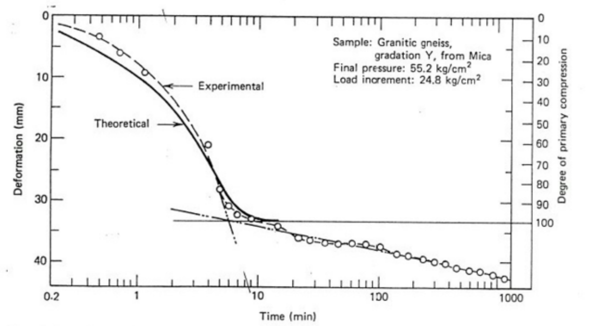
A cause de la variabilité de parametres, l'évolution de tassements secondaires a étaites suivies sur une periode de 11 ans dans quelques cents points sur la surface de remblais et sur fondations á Kollsnes. Les resultats de tassements mesureés confirment les hypothèses de calcul et permet le contôle de parametres.

1 ROCK FILL CREEP SETTLEMENTS

It is well recognized and illustrated by settlement measurements of rock fills that the grain structure of such fills does not react instantaneously to changes in stresses although there is no time lag caused by the dissipation of excess pore pressures.

The study of creep settlements in rock fills shows (as illustrated in Fig.1) that there is an initial phase where settlements increase with log time along a "S-shaped" curve (diffusion phase) and a subsequent phase where settlements increase linearly with log time (linear phase), see for instance Marsal, (Marsal, 1973). The "S-shape" curve observed in settlement-time curves of rock fills is attributed to the presence of high contact stresses between corners of neighbouring particles. At some of the contact points, where the contact stresses exceed the strength of the material, failure occurs and, as a result, due to contact flattening and rearrangement of particles, some deformation takes place. The unbalanced contact force at the contact carrying originally high stresses is redistributed to the neighbouring contacts, increasing their stresses which, in turn, may become larger than the material strength and cause new deformation increments. The process continues towards a more uniform distribution of contact stresses between particles. The number of unbalanced contacts will, however, decrease as the redistribution process goes on and the deformation increments will be smaller and smaller (linear phase in log time plot). The settlement magnitude and rate depend on the size, shape and constituent material of the particles as well as on the applied stresses (both mean and deviatoric stresses). Marsal (Marsal, 1965) analyzed the process by stochastic methods and found that the process is governed by Fokker-Plank diffusion equation.

A simplified equation to describe the creep settlements of rock fills is proposed in the present paper. The method is incorporated into computer programs to calculate primary (elastic) and creep settlements of foundations on rock fills.



Correlation of theoretical and experimental time-settlement curves for Mica granitic-gneiss, gradation Y.

Fig.1. Typical oedometer compression-time curve for Mica granitic-gneiss (after Marsal, 1973)

2 2-D FINITE ELEMENT ANALYSIS OF ROCK FILL CREEP SETTLEMENTS

A procedure to calculate creep settlements in rock fills is developed and incorporated into the finite element program **SSCREEP** (Athanasiu, 1991). The procedure is based on the following assumptions:
1) The creep strain increments are defined by the following stress-strain-time law:

$$\Delta\varepsilon_{vollc} = \beta_{vol} \cdot \sigma_m \cdot \log_{10}(\frac{t}{t_o}) \tag{1}$$

$$\Delta\varepsilon_{shlc} = \beta_{sh} \cdot \frac{\sigma_d}{\sigma_{df}} \cdot \log_{10}(\frac{t}{t_o}) \tag{2}$$

where $\Delta\varepsilon_{vol\,lc}$ and $\Delta\varepsilon_{sh\,lc}$ are volumetric and shear strain increments from time t_0 to time t during the linear phase of the creep process, respectively; β_{vol} and β_{sh} are creep parameters for volumetric and shear creep strains, respectively; σ_m, σ_d and σ_{df} are mean (octahedral) effective stress, deviatoric stress and failure deviatoric stress, respectively.

$$\Delta\varepsilon_{voldc} = 2 \cdot \frac{\beta_{vol}}{c \cdot r_t} \cdot \sigma_m \cdot \tan^{-1}\left[e^{c \cdot \log_{10}\left(\frac{t}{t_m}\right)} \right] \tag{3}$$

$$\Delta\varepsilon_{shdc} = 2 \cdot \frac{\beta_{sh}}{c \cdot r_t} \cdot \frac{\sigma_d}{\sigma_{df}} \cdot \tan^{-1}\left[e^{c \cdot \log_{10}\left(\frac{t}{t_m}\right)} \right] \tag{4}$$

where c and r_t are empirical constants that can be determined by the technique of best fitting the "S-shaped" diffusion creep settlement curve and t_m is the time of the inflexion point of the curve.
2) The creep displacement pattern (relative magnitude and direction) is similar to the pattern of the displacements from body forces (own weight of the soil). Using this assumption the creep strain increment components in x, z and xz directions can be computed as :

$$\Delta\varepsilon_{cx} = \frac{\Delta\varepsilon_{volc}}{2} + \frac{\Delta\varepsilon_{shc}}{2}\cos(2\theta)$$

$$\Delta\varepsilon_{cz} = \Delta\varepsilon_{volc} - \Delta\varepsilon_{cx} \tag{5}$$

$$\Delta\gamma_{cxz} = \Delta\varepsilon_{shc} \cdot \sin(2\theta)$$

where $\Delta\varepsilon_{volc}$ and $\Delta\varepsilon_{shc}$ are volumetric and shear strain increments in either diffusion or linear creep phase; θ is the orientation angle of principal (maximum) strain from horizontal (x-axis)
3) The effect of creep strain increments on elastic soil moduli can be neglected.

The finite element procedure consists of the following steps:

1) Calculate equivalent nodal forces from body (own weight) forces of the rock fill and perform an own weight analysis to find the strains and stresses from gravity forces. The Poisson's ratio used in this analysis is calculated from the coefficient of earth pressure at rest, ko:

$$\nu = \frac{k_o}{1 + k_o} \tag{6}$$

2) For each time interval calculate the vector of creep strain increments, $\{\Delta\varepsilon_c\}$, as described in eqs. (1) to (5) using the mean and deviatoric stresses calculated in step 1. The equivalent creep stress increments and nodal force increments due to creep are then calculated as:

$$\{\Delta\sigma_c\} = [D] \cdot \{\Delta\varepsilon_c\} \tag{7a}$$

$$\{\Delta F_c\} = \int [B]^T \cdot \{\Delta\sigma_c\} \cdot dV \tag{7b}$$

where [D] is the elastic stiffness matrix of each element, $\{\Delta\sigma_c\}$ is the vector of equivalent creep stress increments, {ΔFc} is the vector of equivalent nodal point forces due to creep and [B] is the strain-displacement transformation matrix.

3) For each time increment perform a "creep analysis" , i.e. solve for displacements due to equivalent creep nodal forces, {ΔFc}, calculated in step 2.

The creep parameters used with finite element procedure were first calibrated against measured creep settlements at the dynamically deep compacted rock fill for Post Terminal Building at Solheimsvannet in Bergen, Norway. The parameters were thereafter used for preliminary analyses of creep settlements of rock fill atTroll Onshore Plant, Kollsnes, Norway. Typical results from preliminary analyses are shown in Fig.2 showing the influence of a shell sand layer under the rock fill and the barrier on creep settlements at rock fill surface.

3 SIMPLIFIED 3-D ANALYSIS OF PRIMARY AND CREEP SETTLEMENTS OF FOUNDATIONS ON ROCK FILLS

In connection to foundation detail design at three large Onshore Plants on rock fills, including Troll Onshore Plant at Kollsnes (AS Norske Shell), Snøhvit LNG Terminal at Hammerfest (STATOIL) and Ormen Lange Onshore Terminal at Aukra (Norsk Hydro) in Norway, the calculation of elastic and creep settlements of large number of foundations constructed on or in the rock fill, subjected to different loads and interacting with each other required 3-D settlement analyses. A simplified method was derived using theoretical assumptions described in Chapter 2 and incorporated in a Visual Basic program, **RockFill** (Athanasiu 2000), for 3-D analyses of settlements.

The method calculates the settlements by summing up the vertical creep strains in the rock fill along vertical lines. The stresses distribution along vertical lines from foundation loads are calculated using the Theory of Elasticity and are superposed to the stresses from own weight of the fill. The simplified 3-D procedure is described as follows.

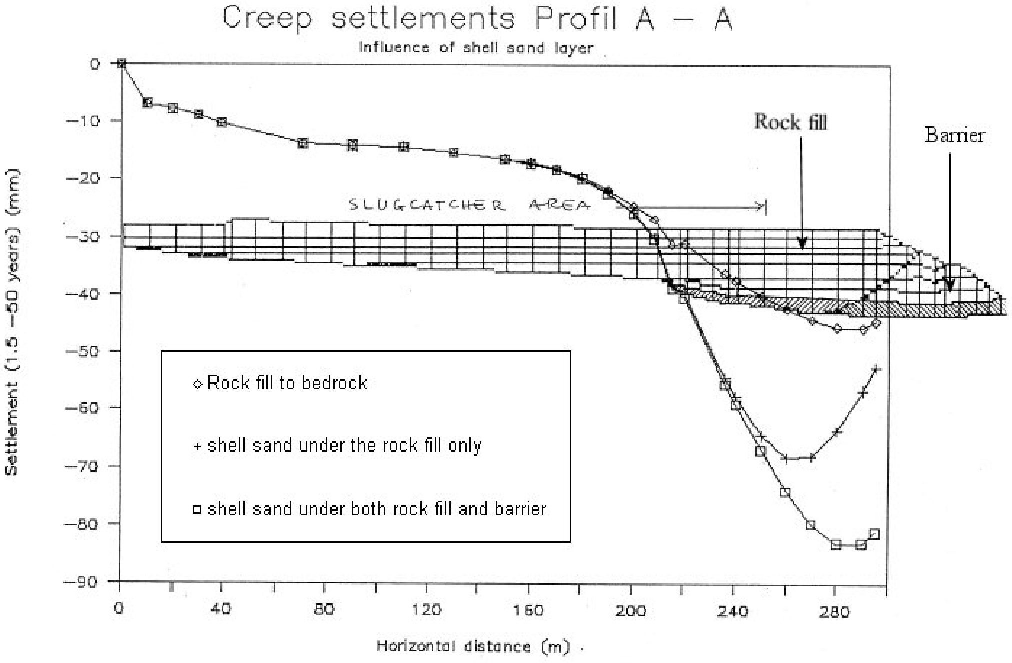


Fig.2. Result from finite element creep analyses using the program **SSCREEP**

3.1 *Rock fill creep settlements*

The linear creep settlement of a point in or on the rock fill can be estimated as:

$$s_{cc} = \beta \cdot s_f \cdot A_\sigma \cdot \log_{10}(\frac{t}{t_o}) \tag{8}$$

where: s_{cc}– creep settlement during linear phase (linear increase of settlement with log time);
β - creep parameter for rock-fill material; s_f – bed-rock slope factor:

$$s_f = 1 + \frac{\tan\alpha}{1+\tan^2\alpha} \cdot (1 - K_o) \tag{9}$$

α -the slope of bed-rock; k_o – earth pressure coefficient at rest; A_σ - area of the effective vertical stress under the level of observation and down to the bed-rock; t- the time elapsed from the completion of the rock-fill up to observation level; t_o- reference time;
The diffusion creep settlement is calculated as:

$$s_{cd} = \frac{2 \cdot \beta_o}{c} A_\sigma \tan^{-1}\left[e^{c \cdot \log_{10}\left(\frac{t}{t_m}\right)} \right] \tag{10}$$

where: c – constant β_o – maximum slope of diffusion curve (at inflection point); t_m – the time corresponding to inflection point on diffusion settlement curve; The ratio between the diffusion settlement, s_{cd}, and the settlement along the creep line at the same time, s_{cc}, is defined as the degree of diffusion process, R:

$$R = \frac{2 \cdot \beta_o}{c \cdot \beta \cdot \log(\frac{t}{t_o})} \tan^{-1}\left[e^{c \cdot \log_{10}\left(\frac{t}{t_m}\right)} \right] \tag{11}$$

The time at which s_{cd} equals s_{cc} is called diffusion time, t_d

Both t_d and t_o can be obtained from the following two conditions:

-the slope of the diffusion curve at time td equals the slope of the straight line:

$$\log\left(\frac{t_d}{t_m}\right) = \frac{1}{c} \ln\left[\frac{1}{r_t}\sqrt{(1 - r_t^2)}\right] \tag{12}$$

-the diffusion settlement equals the creep settlement:

$$c \cdot \log\left(\frac{t_d}{t_m}\right) = \ln\left[\tan\left(\frac{r_t \cdot c}{2} \cdot \log\left(\frac{t_d}{t_o}\right)\right)\right] \tag{13}$$

The creep parameters for normally or dynamically, deep compacted rock fills are determined from experience with rock fills and dam constructions. Fig.3. shows the development of rock fill settlements with time at Kollsnes Onshore Plant (Stordal.et.al.,1995). The Kollsnes rock fill was dynamically deep compacted and the measurement points are placed at different levels on or in the rock fill. The measurements are "normalized" with respect to the area of the stress diagram, A_σ, under the measurement level so that, theoretically, all the measurements should lay on the same curve reflecting the creep properties of the rock fill material. The plots of upper and lower bound curves resulted from measured settlements of rock fill dams (as shown in Fig.4) are also plotted on Fig.3. It is clear that the upper and lower bound curves show a larger range of creep settlement variation for rock fill dams than for dynamically deep compacted rock fills. The explanation for this difference is that generally the rock fill dams have a trapezoidal cross section form where the stresses from own weight of the rock fill induce a higher shear strength mobilization degree than for the case of large flat rock fills such as the rock fill at Kollsnes. In addition the differences may be attributed to the different compaction methods.

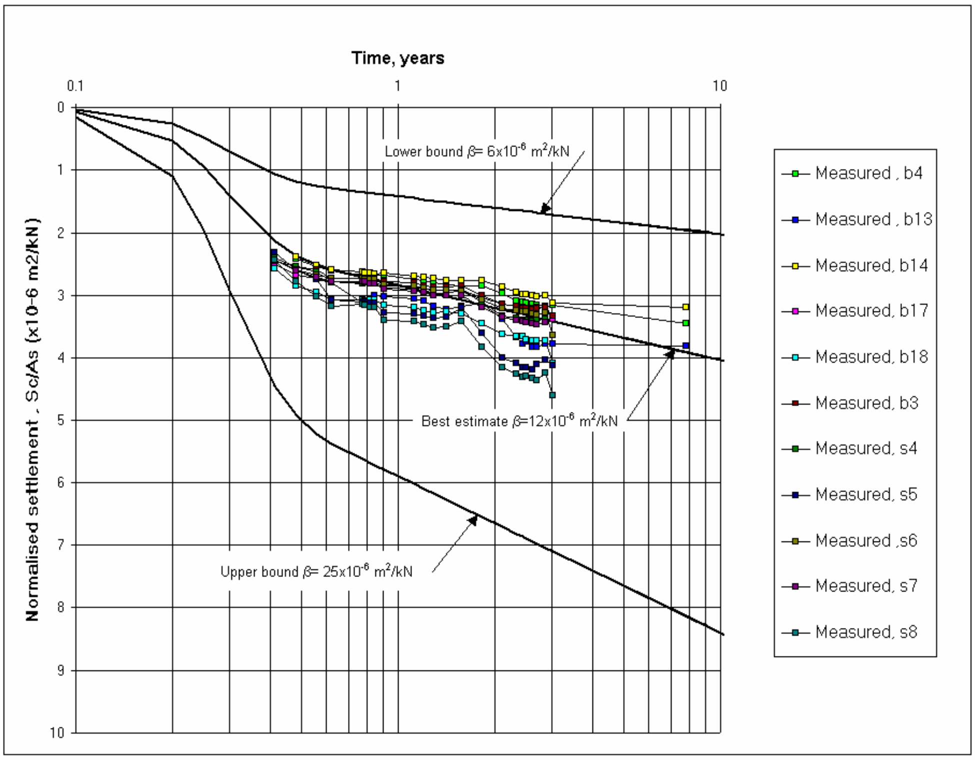


Figure 3. Comparison of settlement measurement at Kollsnes with calculation model using $\beta_{\text{lover bound}}$, $\beta_{\text{best estimate}}$ and $\beta_{\text{upper bound}}$.

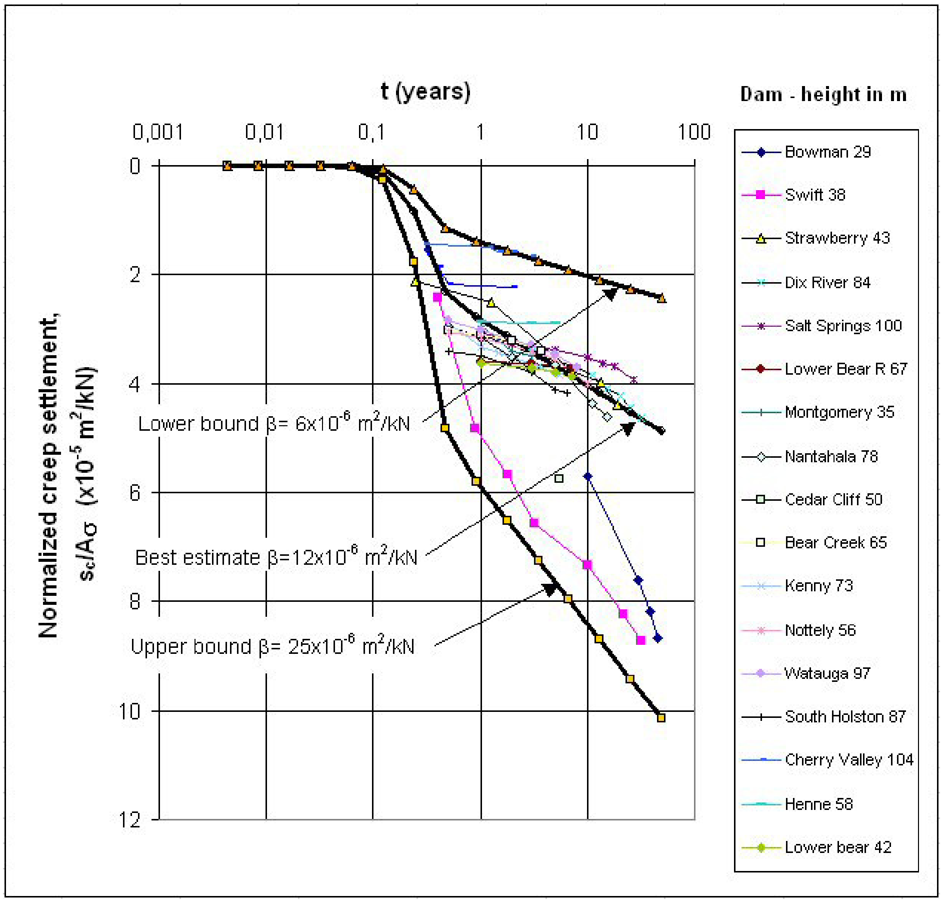


Figure 4. Comparison of settlement for 17 different rock fill dams with calculation model using $\beta_{\text{lover bound}}$, $\beta_{\text{best estimate}}$ and $\beta_{\text{upper bound}}$.

3.2 *Settlements of foundations on rock fills*

If a foundation is placed (and loaded) at a time t_f after the completion of the rock fill, it will follow the rock fill settlements from the time t_f to the time t_{life}, where t_{life} is the life time of the rock fill. In addition, the stresses induced by the foundation loads will cause elastic and additional creep settlements of the fill.
Total settlements at a foundation location are calculated as the sum of the following components (Fig.5):

1) creep settlement due to own weight of the rock fill :

$$s_{fc1} = \beta \cdot \log(\frac{t}{t_o}) \cdot A_\sigma \cdot f_{red} \tag{14}$$

where f_{red} is a reduction factor accounting for the time elapsed between completion of the rock fill and application of the foundation loads, t_f, and t_{life} is assumed the life time of the construction.

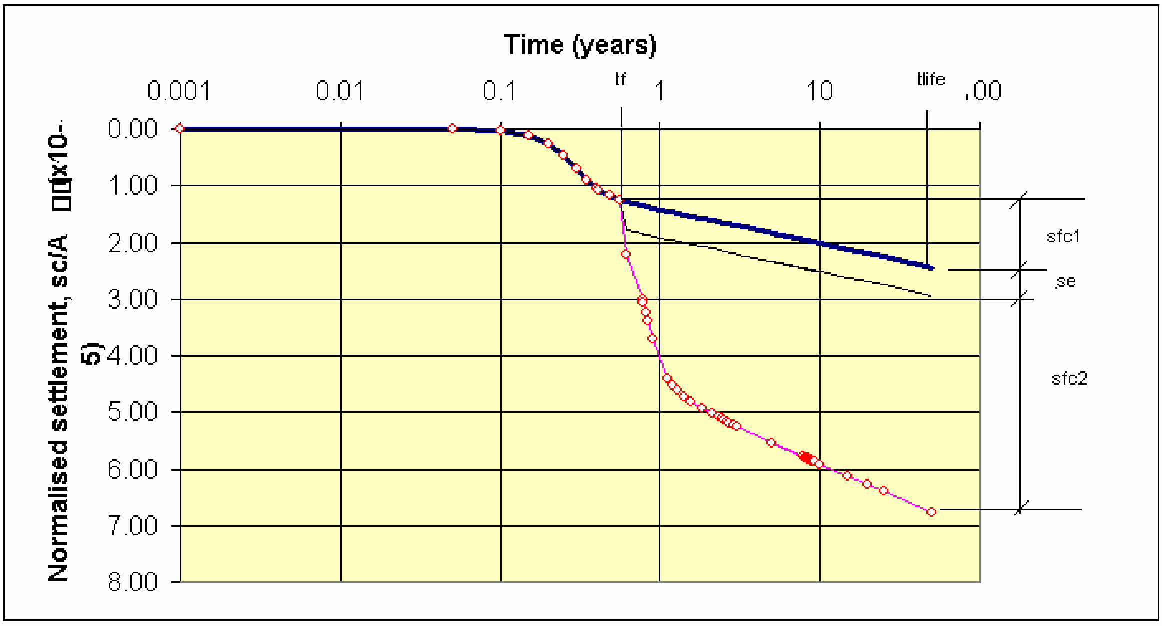


Fig. 5. Components of foundation settlements

$$f_{red} = 1 - R(t_f) \cdot \frac{\log\left(\frac{t_f}{t_o}\right)}{\log\left(\frac{t_{life}}{t_o}\right)} \tag{15}$$

where $R(t_f)$ is the degree of diffusion of rock fill creep settlements, at the time t_f.

2) creep settlement due to stresses induced by applied foundation loads

$$s_{fc2} = \beta \cdot \log(\frac{t_{life}}{t_o}) \cdot A_{\sigma f} \tag{16}$$

$A_{\sigma f}$ is the area of the stress diagram due to foundation loads; it can also include the influence of neighbouring foundations.

3) primary (elastic) settlement due to stresses induced by applied foundation loads are calculated using the solutions of Theory of Elasticity for evenly distributed load on elastic half space :

$$s_{el} = \frac{1-2*\upsilon*k_o}{E} \cdot \sum[\Delta\sigma \cdot \Delta z] = \frac{1-2*\upsilon*k_o}{E} \cdot A_{\sigma f} \tag{17}$$

where: ν is Poisson's ratio, E is elasticity modulus, Δσ - vertical stress at a depth z, due to even distributed foundation load over a rectangular area B*L, $A_{\sigma f}$ - is the area of stress diagram in a selected point due to foundation loads, down to bedrock.

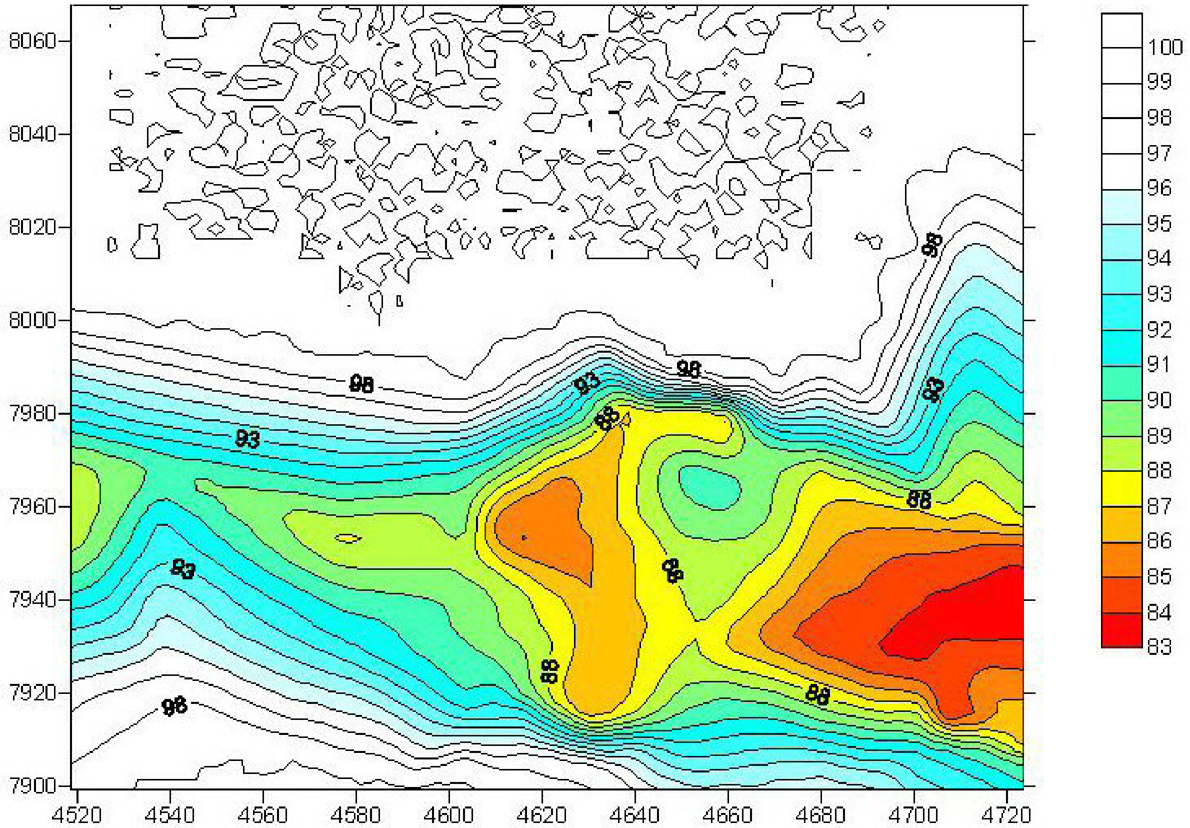
In view of relative good agreement between measured and calculated settlements both at Kollsnes and at rock fill dams (as shown in Figs. 3 and 4), it is recommended to use the following parameters:

For the linear phase of creep settlements : t_o=0.01-0.001 years (t_o=4.35*10^{-3} years); $\beta_{lower\ bound}$=6*10^{-6} m^2/kN; $\beta_{best\ estimate}$=12*10^{-6} m^2/kN and; $\beta_{upper\ bound}$=25*10^{-6} m^2/kN.
For diffusion phase: t_m=0.3 years; c=6.75, r_t=0.2, with $\beta_o=\beta/r_t$

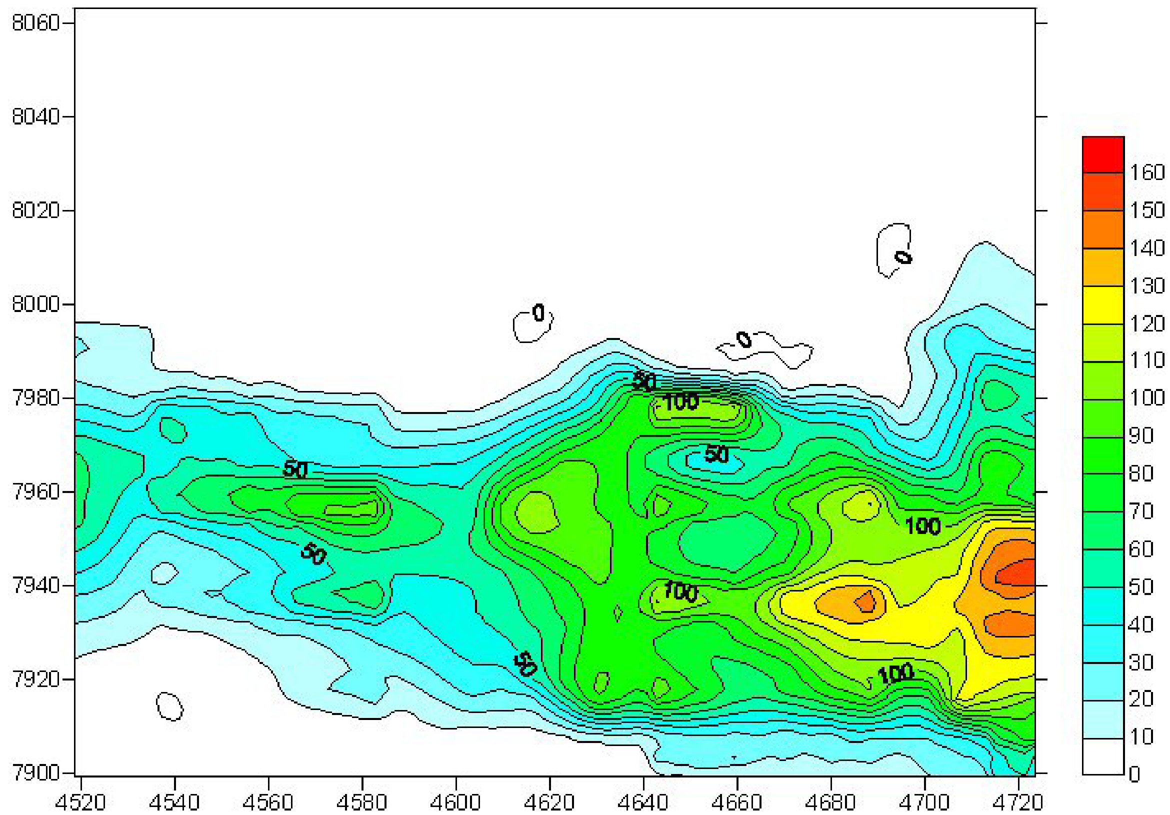
4 SIMPLIFIED 3-D ANALYSES OF FOUNDATION SETTLEMENTS

The method described in Sections 3.2 and 3.3 is incorporated in a Visual Basic program, **RockFill.** The program calculates the vertical stresses induced by foundation loads and those from own weight of the rock fill and calculates the settlements at diferrent time intervals according to eqs. (14) to (17). The settlements within the rigid foundation areas are corrected so that the settled surface should be a plane translated vertically and rotated about the two axes x and y. The translation and rotations are obtained from the condition that the settlement volume and moments about the two axes must be equal to the corresponding volume and moments of the "flexible surface" settlements. The results are presented as color plots of settlement contours at the desired time interval.
The use of the program is illustrated by settlement analysis of Slug Catcher area at Ormen Lange Onshore Terminal, as shown in Figs. 6,7 and 8.

Figure 6 shows the bed rock elevations varying between +100 and +83 m, corresponding to a rock fill thickness of up to 17 m. Settlement contours in mm are shown in figure 7. The settlements are taking into account both rock fill weight and foundation loads (typically 75 kN/m^2). Figure 8 shows the pipe structure and foundations together with the settlement contours. The plot is used to determine the zones where differential settlements are critical for the structure and to design the foundations accordingly.



Figur 6. Bedrock contour lines, slug cather area.



Figur 7. Settlement results, slug cather area. Scale in [mm].

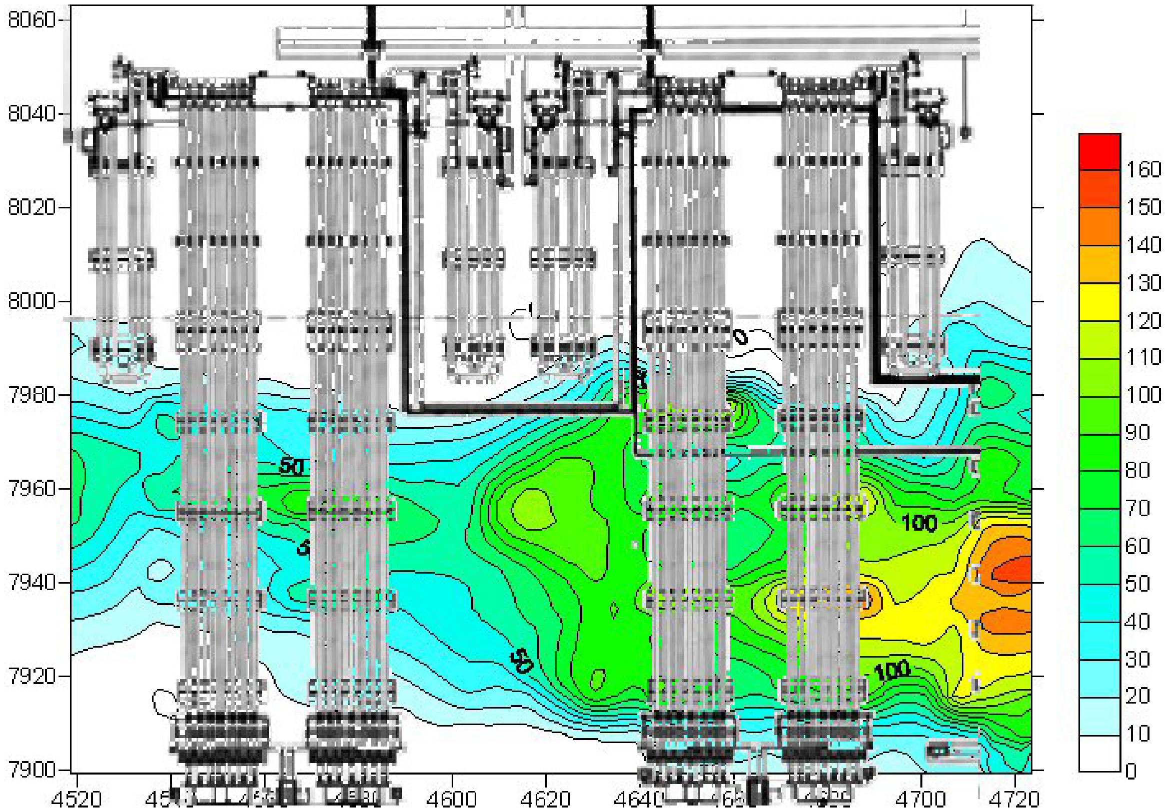


Figure 8. Settlement results in slugcathcer area with pipe structures. Scale in [mm].

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