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History of Passive Pressure of Non-Cohesive Mass and Its Consequences for Theory of Earth Pressure

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HISTORY OF PASSIVE PRESSURE OF NON-COHESIVE MASS AND ITS CONSEQUENCES FOR THEORY OF EARTH PRESSURE

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ABSTRACT

Recently, the long-term research of passive pressure has been carried out. The rotation about the toe of the physical model sized 3.0*1.0*1.2 m has brought unexpected results similarly as the previous research of active pressure. The research contains among others the investigation of time stability of both pressure components, i.e. normal component and shear component.

The paper presents latest information about physical modelling and its results and compares the results with the concept of the developed the General Lateral Pressure Theory (GLPT). This theory is supported also by the monitored time instability of lateral pressure, which is discussed as well. The recent state of the General Lateral Pressure theory in contradiction to the state of the recent earth pressure theory is mentioned.

INTRODUCTION

The actual earth pressure theory is based particularly on the works of Ohde, Terzaghi, Caquot-Kerisel, Ehrenberg, Jáky, de Wett, Sowada, Siedek, Myslivec, Pruška, Janbu, Brooker-Ireland, Morgenstern, Eisenstein, Gudehus {1980} and others. Very advanced actual studies, analyses and experiments were presented at the XIIth EC SMGE in Amsterdam 1999, at the jubilee IS JGS in Tokyo 1999, RC in Shanghai 2001 and XVth IC SMGE in Istambul 2001, which used very advanced technologies, and the latest versions of such programs as Ariizumi et al., Das, Kort et al., Onishi & Sugawara, Powderham, Siemer et al., Simpson, Sterling, Taylor, Uchiama,Wu & Prakash and others. The earth pressure computation models use a scale of very different algorithms from the simplest, which have been used for dozens of years, to very advanced algorithms using FEM or BEM. Probably the most widespread method is the Depended Pressure Method (DPM), which probably was used first by Zapletal 1980 and was based on the simple elastic-plastic relation shown in Fig. The weakest points of the DPM are the uncertainty of defining procedures of the elastic constant, the area of pressure at rest and the course of the plastic areas. However, the ancient original idea (probably of French and Belgian fortification engineers) on the effect of solid earth wedges, followed by the idea on the possibility of the effect of the *particular type* of earth pressure against the *whole* retaining structure and the idea of the *particular stress/strain state (mobilization of shear strength) in the whole respective part* of the mass in dependence on the movement of the toe or the top of the structure have persevered in regulations, theory and practice.

Objections to the recent theory resulting from its weak points have led to the research of non-cohesive granular masses, concentrating both on physical and numerical modeling.

Two medium-term experiments with active lateral pressure (experiments E1 and E2) of loose sand acting on a retaining wall were performed. The experimental stand makes it possible to measure both the normal and the tangential components of pressure. The experiments showed some rather unexpected behaviour of the granular mass, especially its deformations and failures during three different wall movements. This was the reason of the experiment repetition. The measurements included both components of the pressure of the mass. Two analogous numerical model experiments were made, based on the General Lateral Pressure Theory (GLPT).

Recently, the long-term research of passive pressure (experiment E3) has been carried out. The rotation about the toe of the physical model sized 3.0*1.0*1.2 m has brought unexpected results similarly as the previous research of active pressure. The research contains among others the investigation of time (in)stability of both pressure components.

The paper shortly guides through the recent state of the General Lateral Pressure theory in contradiction to the state of the recent earth pressure theory where the case history has been beginning. This theory is supported also by time instability of lateral pressure, which can be discussed as well. The paper presents latest information about physical modelling and its results and compares the results with the concept of the developed GLPT.

SKETCH OF GENERAL LATERAL PRESSURE THEORY

The problem of lateral (earth) pressure can be characterised according the approaches to four basic forms of the pressure or the states in the granular (soil) mass, i.e.:

- pressure at rest,
- acting of the extreme pressure values (active and passive),
- intermediate values and
- residual pressures (active and passive).

GLPT is the non-conventional theory and the presented research of passive pressure is one part of proving this theory. A sketch of the theory appears useful for its consequences.

Pressure at Rest

The lateral pressure at rest originates under the condition of zero or very small movements of the retaining structure whether they head away from the granular mass (active pressure at rest) or into the mass (passive pressure at rest). The value of the pressure at rest for the same granular material may vary within an interval appropriate for the given material. For the magnitude of the limit values of this interval in *non-cohesive* materials and for the horizontal surface of the granular mass two known formulas were derived by means of the coefficients of the pressure at rest expressing the ratio between horizontal and vertical stresses. The first is the Jáky [3] equation (1a) and its simplified and world-wide extended form (1b) (e.g. by EC7-1, Art. 9.5.2):

$$K_{01} = \frac{1-\sin\phi'}{1+\sin\phi'}\left(1+\frac{2}{3}\sin\phi'\right) = K_{0a} \qquad (1a),$$

$$K_{0a} = (1-\sin\phi') \qquad (1b),$$

The second is Pruška´s [16] formula (2a) applicable to the upper limit of the interval of the pressure at rest (passive pressure at rest) and its simplified form (2b) :

$$K_{02} = \frac{1+\sin\phi'}{1-\sin\phi'}\left(1-\frac{2}{3}\sin\phi'\right) = K_{0p} \qquad (2a),$$

$$K_{0p} = (1+\sin\phi') \qquad (2b).$$

Then equation

$$K_0 = (1-\sin\phi')\sqrt{R_{0c}} \qquad (3),$$

gives the initial value of the pressure at rest, where $K_{0a} \le K_0 \le K_{0p}$ and where R_{0c} is the pre-consolidation number. According to Koudelka´s amendment the formula should be used for the values of $\le (1+\sin\phi') / (1-\sin\phi')$.
The interval limits for cohesive soils may be modified by the influence of cohesion with the parameters of shear strength, which can be obtained from the Myslivec´s formulae

$$\phi_0' = arc\ sin[\sin\phi' / (2-\sin\phi')] \qquad (4),$$

$$c_0' = c' * tg\,\phi_0' / tg\,\phi' \qquad (5).$$

These parameters may to be used for the earth pressure calculation of Rankin´s stress state.

This concept leads mathematically to a singularity for the zero value of retaining structure movement. In fact the singularity is not possible but the press value acting against a rear face point of the perfect rigid structure can be different due to a number of conditions. The GLP theory supposes (see Fig. 1) that the initial lateral press value $K_0\ \sigma_1$ should be in the interval limited by the coefficients $K_{0a} \le K_0 \le K_{0p}$

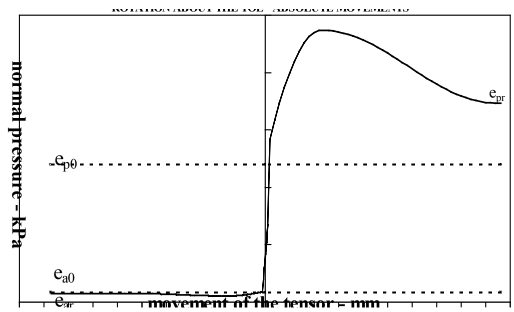

Fig. 1. Theoretical relation between the normal component of lateral pressure and the structure rare face point in the depth of 0.265 m, or in the location of No.2 tensor.

For cohesive materials it is advisable to modify the limits of this interval with the influence of cohesion (Myslivec [14]). The actual value of the pressure at rest affecting the given point of the retaining structure in the given time depends on the position of that point, actual conditions (geological composition, geotechnical soil characteristics, movement of the structure, compaction, etc.) and on the history of the stress/movement relation of the structure or the stress/strain relation in the granular mass.

Extreme Pressure Values

It is no doubt that extreme (active and passive) lateral pressures exist for each point of a structure under mass surface. On the other hand it is very improbable or, perhaps impossible, that a extreme lateral pressure acts in general along whole structure, except of the very special case of value combination of movement and deformation of structure and granular mass and their parameters. Let us leave our interest in numerical values of the extreme pressures and let us deal with behaviour of the mass, the contact between mass and structure and with the function of shear strength mobilization.

The Role of Extreme Pressures. During general movements and deformations of the retaining structure in any direction (out of the granular mass or into it) only a certain part of the granular (soil) mass, which we can call its active part, may co-operate and be deformed (see Fig. 2b). During a partial spot movement or of its component these changes will take place in a limited zone (activated zone) of the granular mass, maximally along the whole length of its active part (see Fig. 2a). Of course, mass deformations caused in unitary activated zones are added and they influence together, so the active part of the mass originates. The co-operation of the granular mass and its deformation generates a change of its general stress state, particularly of the state of shear stress (shear strength mobilization). The degree of shear strength mobilization is not the same due to different displacements on different critical shear surfaces of various spots (see Fig. 2). The values of the respective movements (which can be called critical or extreme or peak) of various structure spots, generating extreme (peak) pressures applied to them, differ and depend particularly on the movement of the structure in the given spot even on the deformations and stress state of the activated zone of the granular mass. The direct simple dependence of mobilization of peak shear strength on an unitary relative limit movement value (see e.g. EC7-1, Annex C.2, Tab. C.1 and C.2 or ČSN 73 0037, Fig.15) in the whole active part of mass does not correspond to reality and a supposition of the acting of pressure in extreme values (active or passive) is not correct.

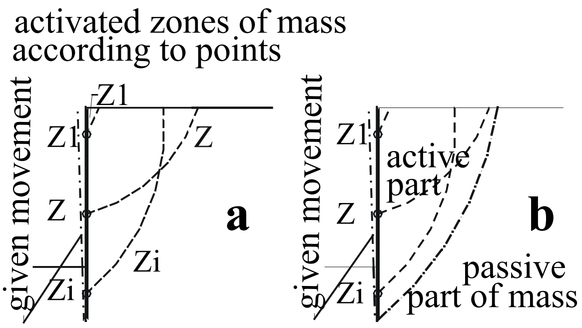

Fig. 2. Different acting of granular mass on different structure spots and corresponding different mobilization of its shear strength on critical shear surfaces.

The extreme (peak) values in the given spot of the retaining structure are passive or active pressures which occur when the shear strength of the respective part (shear surface) of the soil mass is fully mobilized and there is no impediment to the necessary type and amount of movement of the ground or of the wall. The constraint of the retaining structure, struts, anchors or similar elements impose kinematical conditions for the structure and they yield only *possible* lateral pressure values and distribution and *not necessarily* the most favourable (or economical) extreme (peak) values and distribution of earth pressures.

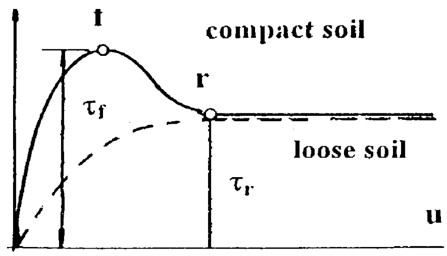

Fig. 3. Relation of shear strength τ to displacement u for compact soil (solid) and loose soil (dashed).

Substantiation. The present concept of the application of the extreme (peak) values of earth pressure applies these values to the whole length (depth) when the limit value of corresponding movement of the three basic movement types has been obtained. The limit movement value to mobilize limit earth pressure is defined as the relative value of movement with regard to the depth of the structure. This concept is at variance with reality and this fact can be proved by the following experience and results :

a) identical critical displacements for the extreme (peak) shear strength of identical material samples during shear tests of soils (Fig.3),
b) geometric and static conditions of the retaining wall movement (Fig.5 and 6),
c) soil mass deformations (Fig.4),
d) results of physical experiments Nos. E1 and E2/1, 2, 3 - Lateral pressures of a loose non-cohesive granular mass No. 1/Part 1: Rotation about the toe, Part 2 - Rotation about the top, Part 3 - Translative motion; results of numerical experiments N1 and N2/1, 2, 3 (Grant Agency of the Czech Republic - Grants No. 103/0702/97 and 103/0632/98, see (11),(13),(14),(16),(19).

As regards the facts sub a,b,d), the shear tests, the geometric and static conditions of the movement and the results of the above mentioned experiments yield the following consequences. The displacement necessary for the mobilization of the peak shear strength value u_f (see Fig.3) of a certain soil is the same for the same material, provided the influence of the non-homogeneity of the material and the test inaccuracies and deviations are eliminated. This fact applies to the whole homogeneous mass.

Consequently, the mobilization of shear friction in a general point of the granular mass depends on the magnitude of the shear displacement in this point. The displacements in the respective part of the activated zone of the granular mass depend on the magnitude of the of the corresponding point movement of the contact structure surface (rear face) and the granular mass. The value of the movement of the contact surface depends not only on the movement of the structure as a whole (as a rigid element), but also on the deformation of the structure. The different values of the movement of the

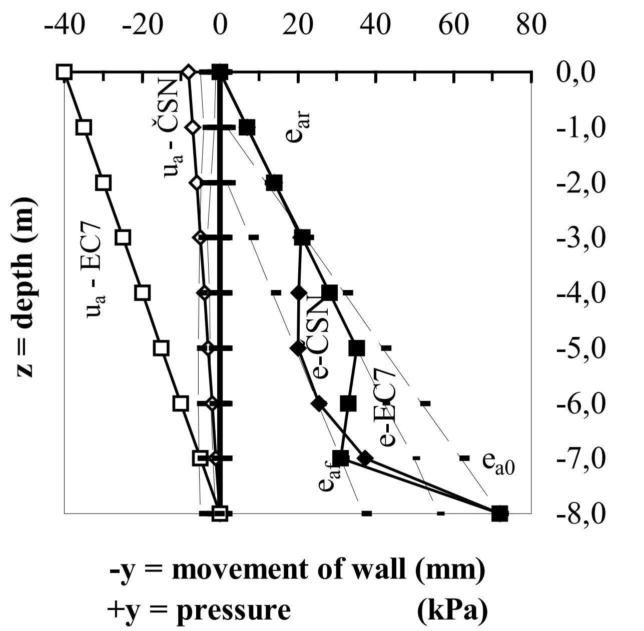

Fig. 4. Active pressure of consolidated soil acting on retaining structure during the standard limit movements for rotation about the toe after EC 7-1 and ČSN 73 0037.

different points of the structure generally cannot give rise to identical displacements in the corresponding parts of the activated zone of the granular mass (see Fig.4).

As to the fact sub c), the active part can co-operate and be deformed during *any general* movements and deformations of the retaining structure in any direction (out of the granular mass or into it). During a *partial* movement these changes will take place in a more limited part of the mass, i.e. the *activated zone* of the mass, but maximally in the whole active part. The interaction of the granular mass with the retaining structure and its deformations brings about the changes of the stress state in the mass, particularly of the shear stress state (mobilization of shearing resistance). The scope of shearing resistance mobilization in the activated zone of the mass in a general case is not the same. The values of the respective critical (extreme-peak) movements of the different rear face points producing extreme (peak) pressures on structure mutually differ and depend particularly on the movement of the structure in the given spot and, also on the deformations and stress state of the *whole* activated zone of the mass. For instance, a rotation of retaining structure about the toe affects due to the geometrical conditions that the pressure at rest is acting against the lower part of the structure differently from other parts of the structure (Fig. 4).

The assumption of the identical displacement differences on shear surfaces in the active part of the granular mass, consequently, is at variance with the geometric and static conditions of the deformation of the granular mass particularly during the rotation of the retaining structure and its deflection (see Fig. 6 and 7). In these casesthe individual points of the active part of the granular mass are undergoing very different deformations as a result of which they must generate a different degree of shearing resistance mobilization and, consequently, not exclusively the mobilization of the peak shear strength. It is obvious that the *extreme* (peak) values of lateral pressure cannot affect generally the whole retaining structure.

Apart from the structure movements, the mobilization of shearing resistance in the activated zone is influenced necessarily also by the deformation of the granular mass itself ad 4.2c). In other words the movement of the retaining structure and the ensuing change of stress in the contact spot between the structure and the granular mass (on the structure rear face) necessarily will cause a change of the stress state in the granular mass, because they have changed the boundary conditions of the granular mass. It is obvious that the change of the stress state will bring about also a change of the mass deformation. It follows that the mobilization of the shearing resistance in the active part of the granular mass depends not only on the movement and the deformation of the structure, but also on the *deformation of the granular mass itself***.**

The mobilization of the peak shearing resistance in any given point of the activated zone of a granular mass during any movement of the structure, consequently, does not depend *only* on the *relative* limit movement of the structure. The code

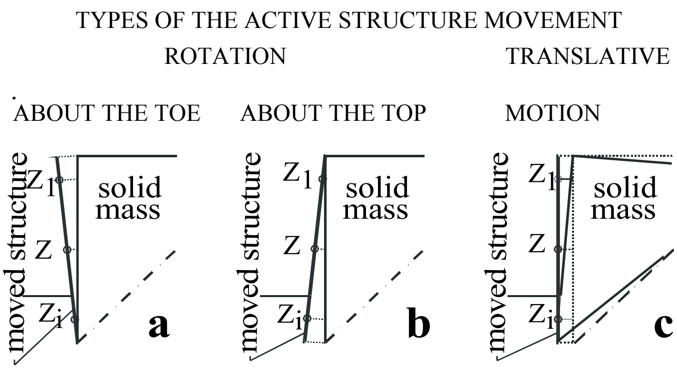

Fig. 5. Geometric and deformation variances of the supposition of solid active part during 3 basic types of active movements.

(standard) *relative* dependence appears to be indirect and not decisive. The mobilisation of the shearing resistance in the given place of the granular mass depends not only on the parameters of the structure itself (rigidity) and its static performance (incl. the movement magnitude and form), but also on the physical characteristics of the granular mass, on the geometry of its activated zone and on the parameters of the contact between the granular mass and the retaining structure.

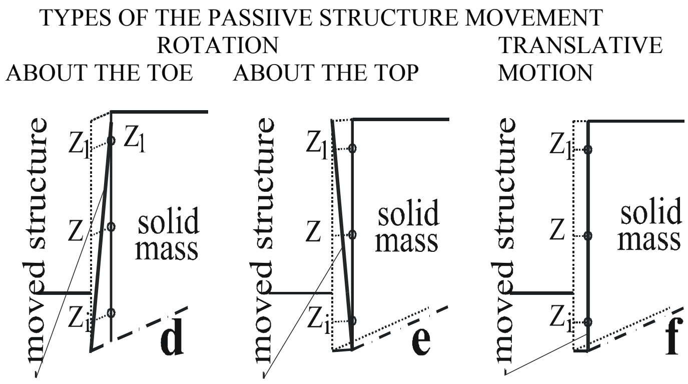

Fig. 6. Geometric and deformation variances of the supposition of solid active part during 3 basic types of passive movements.

The results of an analysis of all six basic movement types of retaining structure (8), i.e. three types in the direction out of the mass (active) and three types in direction into the mass (passive). So the rotation about the toe, rotation about the top and translative motion were analysed by an advanced numerical model based on the GLPT according to the similar relations shown in Figs. 1 and 7.

The original experimental equipment was developed in the Czech Republic, for the research of lateral pressures of granular materials. This equipment with new bi-component pressure tensors, made according to a Czech patent, was used in the experiments E1, E2 and E3. The results of No. E1 experiment with a perfectly non-cohesive granular material have proved the determining influence of the geometric and static conditions of the movement of a rigid retaining structure in the three basic types of the active structure movement, i.e. during the rotation about the toe or the top and translative motion. Every one of these movement types produced a different and highly diverse deformation of the noncohesive granular mass (see (11),(13). This experiment was repeated and its results were confirmed in the practically identical experiment no. E2 (see (13),(19)).

Intermediate Pressures

Let us look at Fig.1 or Fig.7 again. There are a very close section of pressure at rest on both sides of the origin, two extreme values and two residual areas on both margins of the graph. Lateral pressures of values between these limits (active or passive from pressures at rest to residual ones except of extreme pressures) are called intermediate. Two certain intervals of pressure values (both in active and passive side) can define pairs of respective movements, one in the movement interval *before* extreme pressure value and the second in the movement interval *after* extreme pressure value and simultaneously *before* residual pressure areas. The objection ad 1d) concerning residual pressure is beyond the scope of this paper, then let us deal with intermediate pressures only.

The role of intermediate pressures. In current cases a *general* lateral (earth) pressure (active or passive according to the direction of the movement of the structure) acts on the retaining structures, the values of which
correspond with the different rate of shearing resistance mobilization in the individual regions of the activated zone of granular mass. In the case of insignificant or zero changes of the whole structure or its part are exposed to the *general* pressure at rest of a some value within its interval. *Special* case of unitary residual pressure application to the whole structure or its part occurs during its movement equal or greater than the values of their limit residual movements.
The determination of the intermediate value of earth pressure shall take account of the amount of wall movement (deformation including), its direction relatively to the mass and the characteristics of the granular mass (soil) including its stress state. The history of intermediate pressure values plotted against the movement of a spot of retaining structure can be considered similarly but corresponding according to the diagram in Fig.7.

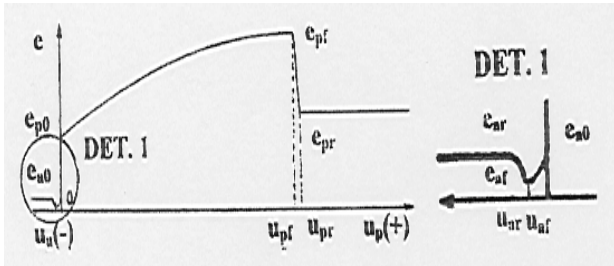

Fig. 7. Dependence of earth pressure value in the given rare face spot of the structure on its movement where:
e_{a0} - active pressure at rest, u_{a0} - limit active movement at rest,
e_{p0} - passive pressure at rest, u_{p0} - limit passive movement at rest,
e_{af} - extreme (peak) active pressure, u_{af} - critical (peak) active movement,
e_{pf} - extreme (peak) passive pressure, u_{pf} - critical (peak) passive movement,
e_{ar} - residual active pressure, u_{ar} - limit residual active movement,
e_{pr} - residual passive pressure, u_{pr} -. limit residual passive movement,

Substantiation. The reasons for the formulation of a more advanced dependence of earth pressure magnitude on the movement of the retaining structure according to Fig.7 and the proofs, that a retaining structure in a standard general case is affected by lateral pressure of general values, corresponding to the various degrees of shear strength mobilization in the individual points (interaction with the structure) of the activated zone of the granular mass, were presented in preceding paragraphs.

The relation proposed in Fig7 means the abandonment of the idea that in a *general* case one certain lateral pressure value (at rest, peak active, peak passive, increased or decreased to a uniform level with reference to peak pressures) can affect the *whole* retaining structure. On the contrary, the proposed relation expresses the opinion that the retaining structure is affected by the lateral pressure of the values corresponding to the *different shearing resistance mobilization in different points of the granular mass* and due to this fact also *different pressures* act on structure. For this reason the lateral pressure affecting the retaining structure can be called *general lateral pressure* (GLP).

Even B. Simpson presented in his lecture (22) at XVth IC SMGE (2001) Terzagh´s diagram (23), which gives a dependence similar to Fig.7 (see Fig.8). However, the conventional *theory* has given no attention to their knowledge.

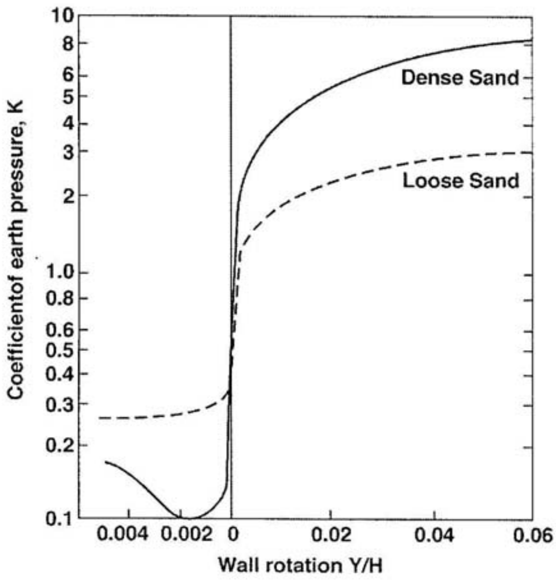

Fig. 8. Terzaghi´s dependence of coefficient of earth pressure on wall rotation Y/H (1943, after B. Simpson 2001).

Residual Pressures

By residual pressure we mean the value of lateral pressure applied to the given point of structure-ground interface, corresponding with the drop of shear strength to residual value τ_r (Fig.4) in the reapective decisive part (critical surface or region) of the granular mass. In other words the residual pressure at the given point of strucutre is generated by the attainment or exceeding of the limit residual displacement u_r in the respective part of the mass which influences lateral pressure applied to the given point. Depending on the *direction* of the movement of the structure (*away from* or *into* the granular mass) we differentiate *active* or *passive* residual pressure.

The drop of shear stress state value in the granular mass of higher top shear strength τ_f (e.g. compact soils) generates necessarily an unfavouable change of boundary conditions of its support, i.e. the pressure in the structure-ground interface. In case of *active* movement of the structure the value of lateral pressure will *rise* to the *residual active* value, in case passive neovement of the structure its value will *drop* to the *residual passive* value. The neglect of this fact in the conventional theory brings about obviously the greatest part of the risk. In granular masses without significant top shear strength (loose soils in Fig.4) no drop of shear strength τ takes place and the application of conventional theory to them is less risky.

The begining of this unfavourable development in a granular mass corresponds with the moment, when the limit shear deformations u_f , belonging to the top shear strength τ_f , has been exceeded. The consequences of such development during the movement of the structure to the *active* side can be observed distinctly in the measured pressure applied to sensors 4 and 5 in Fig.3. In case of other, less loded sensors, the increase is less distinct, but does exist. The results shown in Fig.3 illustrate the case of a practically ideally non-cohesive mass in which only the effect of reduction of the angle of shearing resistance is effective and the influence of cohesion equals zero.

In case of cohesive granular masses, however, apart from the influence of the difference in the angles of shearing resistance $(\tan\phi'_f - \tan\phi'_r)$ there is also the loss of cohesion c'_f (in terms of effective stress), so that the shear stress drop is greater. The influence of the loss of the top shear strength, naturally, depends primarily on its magnitude, but also on the depth of the given point of the structure-ground interface. The influence of cihesion decreases with depth and for their usual values it is significant at small and medium depths. This paper gives the results of an earth pressure analysis based on the final draft of prEN 1997-1: "Eurocode 7 – Geotechnical Design, Part 1: General rules" from October 2001(EC 7-1).

The magnitude of the risk of *active pressure increase* and *passive pressure decrease* due to the drop of top shear strength is expressed by the ratio of horizontal components of lateral pressure e_{hr}/e_{hf} (residual to extreme during top shear strength mobilization) in the diagrams in Fig.5a,b,c,d. The diagrams are plotted against depth below the surface h and for the parameters $\phi'_f = 10^o, 20^o, 30^o$ and 40^o, $\delta/\phi = 0.5$, $\kappa_\pi = c'/\gamma = 0.2$ *and* 1 , where ϕ is the angle of shearing resistance, δ the angle of strucutre-ground interface friction, κ_π the similarity height according to Hamilton´s similarity coefficient π and $k_{\phi r} = \tan\phi_r / \tan\phi_f = 0.7$.

The value of $\kappa_\pi = 0.2$ approximately with less cohesive soils, the value of $\kappa_\pi = 1.0$ with cohesive soils and the scope of $\phi'_f = 10^o$ to 40^o with minor extrapolations covers pratically all soils. The diagrams for active pressure in Figs.5a,b and those for passive pressure in Figs.5c,d, therefore, make it possible to assess the influence of the drop of the top shear strength on the residual value approx. within the boundaries of the set of usual soils.

The *increase of residual active* pressure as compared with the *active* (minimal) pressure is several times as high for $\kappa_\pi = 0.2$ up to a depth of approximately 1 – 4 m, for $\kappa_\pi = 1.0$

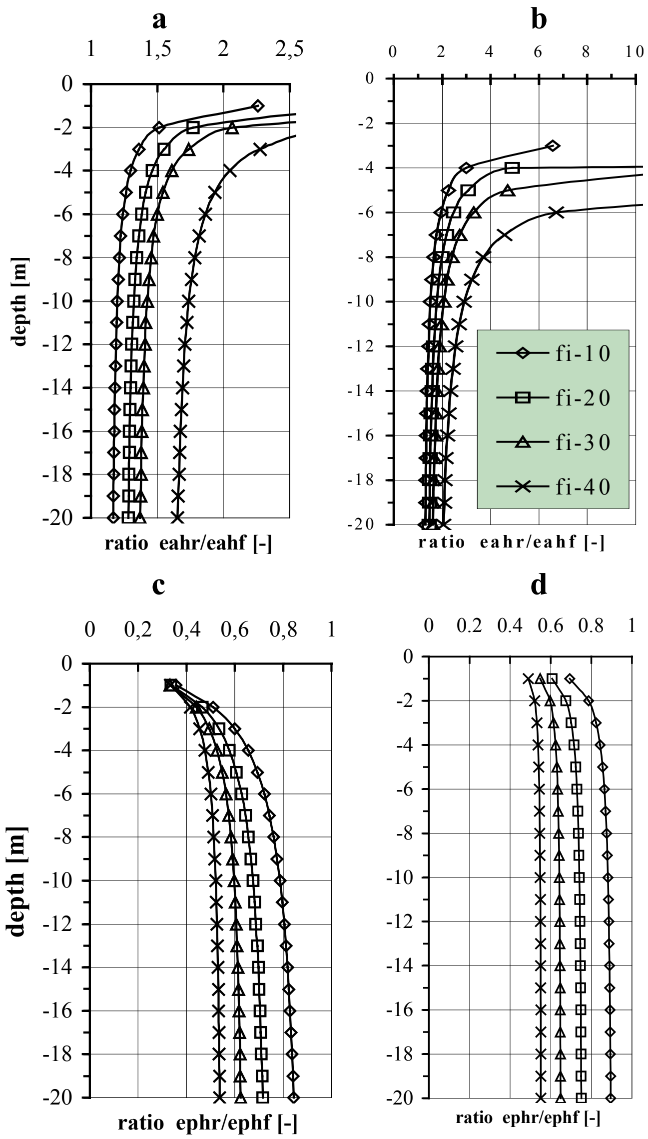

Fig. 9. Ratio the residual lateral pressure e_{hr} and the pressure mobilizing the top shear strength e_{hf} : a – active pressures for $\kappa_\pi = c'/\gamma = 0.2$, b – active pressures for $\kappa_\pi = 1$, c – passive pressures for $\kappa_\pi = 0.2$, d – passive pressures for $\kappa_\pi = 1$.

up to a depth of approximately 5 – 20 m. In greater depths the pressure increases by 100% and less, i.e. by the values of the order of tens of per cent, at a depth of 20 m for $\kappa_\pi = 0.2$ the residual pressure is 17% ($\phi'_f=10^o$) to 65 % ($\phi'_f=40^o$) *higher* and for cohesive soils with $\kappa_\pi = 1.0$ the residual pressure is 30 % ($\phi'_f=10^o$) to 105% ($\phi'_f=40^o$) *higher.*
The *decrease of passive* pressure to the *residual value* as compared with the *maximum* value for the *passive* (top shear mobilization) pressure is greatest also at lower depths, but proceeds less steeply.
Residual passive pressure for $\kappa_\pi = 0.2$ *is 31%* ($\phi'_f=10^o$) to 51 % ($\phi'_f=40^o$) *lower* at the depth of 1 m and 10 % ($\phi'_f=10^o$) to 45 % ($\phi'_f=40^o$) *lower* at the depth of 20 m. In case of cohesive soils of $\kappa_\pi = 1.0$ the residual pressure is 64 % ($\phi'_f=10^o$) to 67 % ($\phi'_f=40^o$) *lower* at the depth of 1 m and 16 % ($\phi'_f=10^o$) to 46 % ($\phi'_f=40^o$) *lower* at the depth of 20 m.

<u>Summary of the GLPT</u>

The matter of the General Lateral Pressure Theory (GLPT) can be summarized in a very simple statement:
1 The acting of the active or passive pressure in extreme value on (whole) retaining structure does not correspond to reality. It can act on the given structure in the very special case only.
2 Lateral pressures in *usual* cases affects structures in *general* values depending on the whole set of parameters and conditions, time including.

VERIFYING OF THE THEORY

The developed GLPT should be verified. Three research projects were proposed and started by the author institute with the support of the Grant Agency of the Czech Republic. The paper presents some important information of the previous physical modeling project (active pressure) and the last one (passive pressure). The results of the numerical modeling project concerning with the comprehensive nonlinear constitutive dependence (Fig.1) were presented earlier (Koudelka 1990, 1992, 1996, 1998a, 1999b). The more detailed information about the experimental procedure see in Koudelka 1999c.

<u>Physical Modeling</u>

A special equipment for the research of bicomponent lateral pressures of multi-phase granular materials was designed and developed during three years 1997-1999 in the Czech Republic by the Institute of Theoretical and Applied Mechanics {Academy of Sciences}. The research should contribute to the verification of the mathematical models (Koudelka 1998a,b, 1999b) and, also, to the drafting of a General Lateral (Earth) Pressure Theory. The equipment has both lateral glass sides for visual monitoring of displacements and deformations in the investigated granular mass. The measurements of lateral pressure are based on the Czech invention of bi-component pressure sensors (Šmíd – Novosad) which enable simultaneous continuous measurements of normal and tangential components as well as dynamic pressures.

The equipment is developed further on. The primary special glass plates of thickness of 10 mm in sides were convenient for experiments E1 and E2 (1998-1999) with *active* pressure. These plates have shown themselves too weak for *passive* pressure during the experiment E3 (2002). Thus, the equipment has been reconstructed (half of 2003) for the glass side plates of thickness of 20 mm. A new engine equipment moving the retaining front wall has been constructed parallely together with a digital monitoring of the front wall movement.

Experiments with active pressure – E1 and E2. At the end of 1998 and in the first half of 1999 the first experiment E1with the mass of a really non-cohesive material (very dry flowing sand) was made. Due to some little expected results the experiment was repeated in the second half of 1999 (E2) exploiting three new more sensible sensors. The dimensions of the tested masses were 1.0 m wide, 1.5 m long and 1.2 m high. The contact surface of the retaining wall was 1.0*1.0 m. The lateral sides of the stand were transparent to enable visual observation of the changes in the mass. The retaining wall has been rigid; it could be arbitrarily moved and its movements were measured by standard mechanical indicators in each corner of the retaining wall. Five bi-component tensors were located perpendicularly to the vertical axis.

Pressure sensing was based on the previously tested and proved bi-component sensors which had been designed and produced especially for this research. These sensors enabled simultaneous continuous measurements of the normal and the tangential (shear) components on the rigid contact surfaces of the tensors. The diameter of contact surfaces was 50 mm. The pressure sensor outputs were processed by a 16-channel BMC amplifier and appropriate hardware and software. The visual observation of deformations and movements within the mass was recorded by a photo camera from a stable position and other suitable points. The sensors are numbered from top to bottom and were placed at the depths of 0.165, 0.365 m, 0.565 m, 0.765 m and 0.965 m below the surface of the mass.

The possibilities of the arbitrary movements of the retaining wall were used for 3 phases of both experiments E1 and E2 with active lateral pressures. One of the three basic movement types was active during each phase. Before the first phase, the experiment E1 with the passive pressure at rest was made by a *passive* rotation about the toe max. 0.11 mm and back only, experiment E2 by a small *passive* translative motion 0.49 mm and back. The experimental equipment and tested material have been described earlier (Koudelka 2000a). Thus, will be stated that the very dry (flowing) sand had following basic parameters : γ = 17.01 kN/m^3 (dense unit weight), $\phi'=43.4^o$ (angle of shearing resistance), $c'= 0$, moisture $w=0.04$ % ..

The mass of E1 consolidated after the first and second phases during the time interval not shorter than one month, the mass of E2 consolidated after the first and second phases during the time interval not shorter than three weeks.. The retaining wall was not moved during the reconsolidation time and thus the mass was influenced by the conditions of the experimental hall only. The next experiments shall follow. The values of each of the movements were 8.75 mm which, considering the height of the wall (1.0m) are higher than the EC7 (final draft 10/2001) in the Annex C, Tab.C.1 as well as ČSN 73 0037 (Earth pressure acting on structures) requirements for the mobilization of maximal shear strength. The phases of the experiment and their relation to standard limit movements for active pressure are shown in the following table:

Fig. 10. Experimental stand with transparent lateral sides. The retaining wall is the blue plane inside the structure on the left. Two red sensors can be seen placed in the retaining wall through the front side. The other one above is not in place

.

Fig. 11. Front space of the retaining wall with 4 bicomponent tensors. The lower is not in place.

Table 1 : Tested movements and standards limit movements for supposed mobilization of active pressure {peak strength}.

Phase	Type of movement	Wall height H	Max. increment Experiments E1, E2 max u	u/H	Standard active limit movements EC 7-1 (final draft 10/2001) u/H	u_{lim}	ČSN 73 0037 u/H	u_{lim}
		m	mm	-	- mm	- mm		
1	Rotation about the toe	1.0	8.75	0.009	0.001-0.005	1 - 5	0.001 -0.002	1 - 2
2	Rotation about the top	1.0	8.75	0.009	0.002-0.010	2 - 10	0.002 -0.004	2 - 4
3	Translative motion	1.0	8.75	0.009	0.0005-0.002	0.5- 2 .	0.0005-0.001	0,5- 1

<u>Experiment with passive pressure – E3.</u> The long-term (one year) experiment E3 tested the granular mass of the same material (dry flowing sand) during the small *active* rotation about the top (*active* pressure at rest - marked phase E3/0) and the long *passive* rotation about the top.(passive pressure – marked phase E3/2). The experiment is described separately, see the experimental equipment in the initial state in Fig. 12.

<u>Time effects on lateral pressure – E3/2T.</u> The experiment part E3/2T with time (in)stability after the *passive* movement of the toe of 8.75 mm took 8 months and 1 week. The experiment is described separately. See the initial state of the phase E3/2T after the toe *passive* movement of 17.5 mm in Fig. 13 .

<u>Numerical modelling.</u> The research has included the numerical modelling of the physical experiments and the development of an advanced computing programme. This theme lays out of the paper theme (see references).

EXPERIMENT WITH PASSIVE PRESSURE – E3/2

At the end of 2001 and during of 2002 the first part of the third experiment E3 was made. The part has been marked E3/2. The physical 2D model consists in a granular mass and a retaining wall, which can perform the movements of all three basic types (rotation about the toe and the top, translative motion) with accuracy lower than 0.024 mm. The wall is 1.0 m high and perfectly stiff, without any deformations of its own. The contact surface of the retaining wall was 1.0*1.0 m. The wall movements were measured by mechanical indicators in each corner of the retaining wall. Five measuring points are situated at the granular mass/retaining wall contact surface 0.065 m, 0.265 m, 0.465 m, 0.665 m and 0.865 m deep.

The lateral sides of the stand were transparent to enable visual observation of the changes in the mass. The granular mass is 3.0 m long, 1.2 m high and 1.0 m wide and consists of the same ideally non-cohesive material (loose very dry sand) like the previous masses. The experimental equipment and tested material described in detail earlier (Koudelka 2000a). Therefore, we shall state merely that the sand had the following basic parameters : γ = 16.14 kN/m^3 (unit weight), w = 0.04 % (water content), ϕ_{ef}' = 48.7^o (angle of the top shearing resistance for low stresses), ϕ_r' = 37.7^o (angle of the residual shearing resistance), c_{ef}' = 11.3 kPa (illusory cohesion), c_r' = 0.

Fig. 12. Lateral view at the experimental stand and the sample of granular mass into. The arbitrary moved front wall is left. State before the experiment E3/2 (passive rotation about the top) on 29th Aug. 2001.

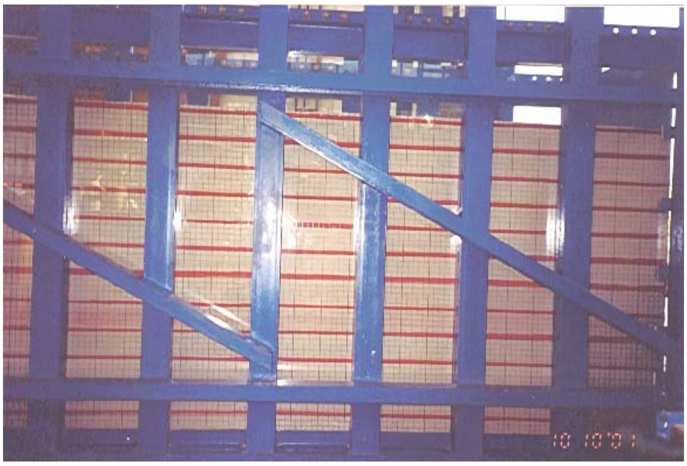

Fig. 13. Lateral view at the experimental stand and the sample of granular mass into after rotation of the toe of 17.5 mm about the top. State during the experiment E3/2 (passive rotation about the top) on 10th Oct. 2001 before the movement stop for the time (in)stability experiment.

<u>Procedure</u>

The possibilities of the arbitrary movements of the retaining wall were used for 3 phases of the previous experiments E1 impossible to investigate all three types of movements on the same sample (model mass) as that used during experiments E1 and E2. Accordingly the experiment E3 deals with rotation about the top only, i.e. phase E3/2.

The mass was slightly compacted by means of the special instrument, which ensured its homogeneity. The whole procedure was designed so as to create an ideal non-cohesive homogeneous mass possible.

Phase E3/2 – Rotation about the Top

The notation of the phase is taken from previous experiments in which rotation about the top was called "phase 2". Before this (first) phase of the experiment, the experiment with the *active* pressure at rest was made by a small rotation about the top of 0.27 mm and back to 0 mm (6th Sept.2001 – E3/2-0). Then the mass was left to consolidate for 32 days and the *passive* part of the experiment began (8th Oct. 2001), the initial part of E3/2 ended on 10th Oct. 2001. The final part of E3/2 began on 18th June 2002 and the final toe movement towards the *passive* side attained about 159 mm on 3rd Dec. 2002.

The state after the final movement can be seen in Fig. 14. The state inside the mass was characterized by the slightly curved major slip surface dividing the *active* mass part from the *passive* one. The *active* part was heavily deformed and further divided on a system of others slip surfaces. The pressure near the rotated wall toe (maximally more of 150 kPa) destroyed the both nearest glass plates, one of them is seen in the Fig. 14. The deformed surface of the mass is shown in Fig. 15.

The retaining wall was not moved continuously but step by step with the periods of reconsolidation between steps. These periods without any movement completed the experiment on the time behaviour. The data of sensors were read and recorded also during the periods of reconsolidation.

Partial Experiment E3/2-T1 with Time Instability

The first movement step E3/2 began 8th Oct. 2001 and the movement of the toe of 15.63 mm was attained after 3 days (10th Oct. 2001). The maximal velocity of the toe movement was approximately of 0.05 mm/min. The following reconsolidation without any movement of the front wall lasted 251 days (until 18th June 2002) and is denoted T1.

The movements of the sensors differed in accordance to their difference to the top of the moved retaining front wall (depth under the surface). The respective (initial and also final during this phase) movements from Sensor 1 to Sensor 5 were *2.84 mm, 5.93 mm, 9.03 mm, 12.12 mm and 15.21 mm*. The first (most important) 60 days (from 10th Oct.2001 to 11th Dec. 2001) are analysed and described in P.Koudelka-T.Koudelka 2003. The analysis proved the time instability of lateral pressure and its analytical formulation.

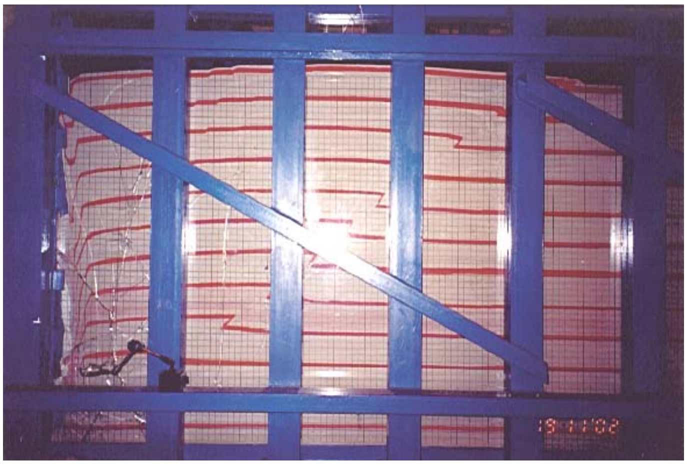

Fig. 14. The state of the mass and the first glass plate near to the moved wall (left) after the toe movement of 134.8 mm before the final movement of 159 mm on 18th Nov.2002. The destroyed glass plate resisted to stress state with the pressure of 150 kPa.

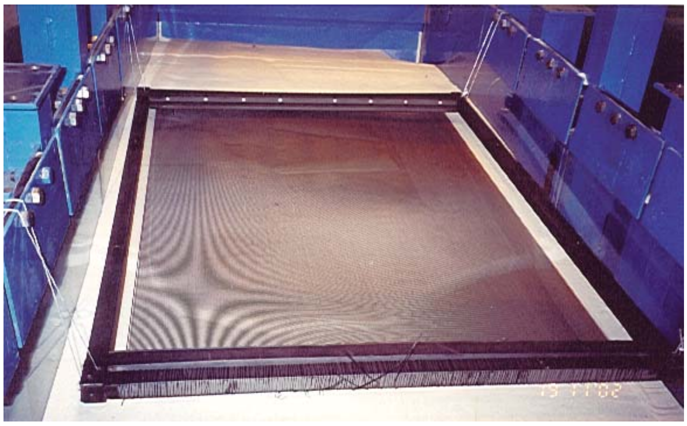

Fig. 15. Deformed surface of the experimental mass from back of equipment after the toe movement of 134.8 mm on 18th Nov.2002 before the final movement of 159 mm. The top of moved front wall is above (blue).

Failures of Side Glass Plates

The first two short and very thin cracks (more or less vertical) in two glass plates nearest to the wall originated after the toe movement about of 13.2 mm on 10th Oct. 2001. The following wall rotation to the value of 17.5 mm caused the extension of the cracks up after the toe movement of 17.5 mm on 10th Oct. 2001. This was the state of the sides at the beginning and during the experiment with time (in)stability. The failure state could not influence the results of this experimental part or not significantly only.

The continuing wall *passive* movement failed the glass plates more and more. The maximal horizontal displacement of the left plate (according to front view) was started at the beginning of the final experimental part after the toe movement of 20.2

mm. Very little displacements (not more than 2 mm) were monitored till the wall movement of 32.9 mm on 27th Oct. 2002. The maximal horizontal displacements of the left plate were monitored according to the movement of the wall toe as follows respectively in mm: 4.45/56.0 on 30th July 2002, 10.01/88.1 on 8th Oct. 2002, 15.03/117.5 on 12th.Nov. 2002, 19.93/141.3 on 25th Nov. 2002 and the final 21.66/159.3 on 3rd Dec. 2002.

Plate displacements on the right side was monitored from the toe *passive* movement of 56.0 mm on113th Aug. 2002. The point with the maximal displacement achieved the value of 2.0 mm after the wall toe movement of 67.1 mm on 30th Sept. 2002, the value of 5.02 mm after the movement of 93.3 mm on 8th Oct. 2002, the value of 10.03 after the movement of 124.5 mm on 18th Nov. 2002, the value of 15.03 after the movement of 149.0 mm and the final value of 17.89 mm after the last movement of 159.3 mm.

The failure process of the glass plates during the second part of E3/2 undoubtedly effected the stress state into the mass. We can consider particularly two effects, i.e. firstly a decrease of stress in the mass and consequently also lateral pressure, secondly a reducing of the mass deformation and displacements on the slip surfaces. The second effect led probably to a drop of the residual shear strength effect.

The magnitudes of the effects were dependent on the displacements of the plates. It appears the effect of the displacement of maximal value of 2 mm is probably little and the maximal displacement more than.5 mm should be considered. The consideration would take into account approximate volume the glass plate deformation which is about 0.08 % of the contiguous mass volume for the maximal displacement of 2 mm, about 0.22 % for 5 mm, about 0.44 % for 10 mm, about 0.66 % for 15 mm. The final displacements had the volume increments of 0.86 % and 0.80 % respectively.

RESULTS

The experiment E3/2 brought a large information which has not been analysed whole yet. The paper presents, with regard to its size some results of the lateral (earth) pressure components. The results of visual monitoring and some others are out of range the paper.

The following diagrams distinct (in their x axis) the toe movement and the absolute movements. The toe movement is defined like the horizontal movement of the centre of the lower wall edge. The toe movement is the same for all sensors. The absolute movement are defined as the horizontal movement of the contact surface centre of the given sensor.

Fig. 16 shows behaviour of both lateral pressure components during the first phase of the experiment, i.e. *active* pressure at rest E3/2-0, and according little passive movements at the initial part of rotation about the top E3/2-1. The total different

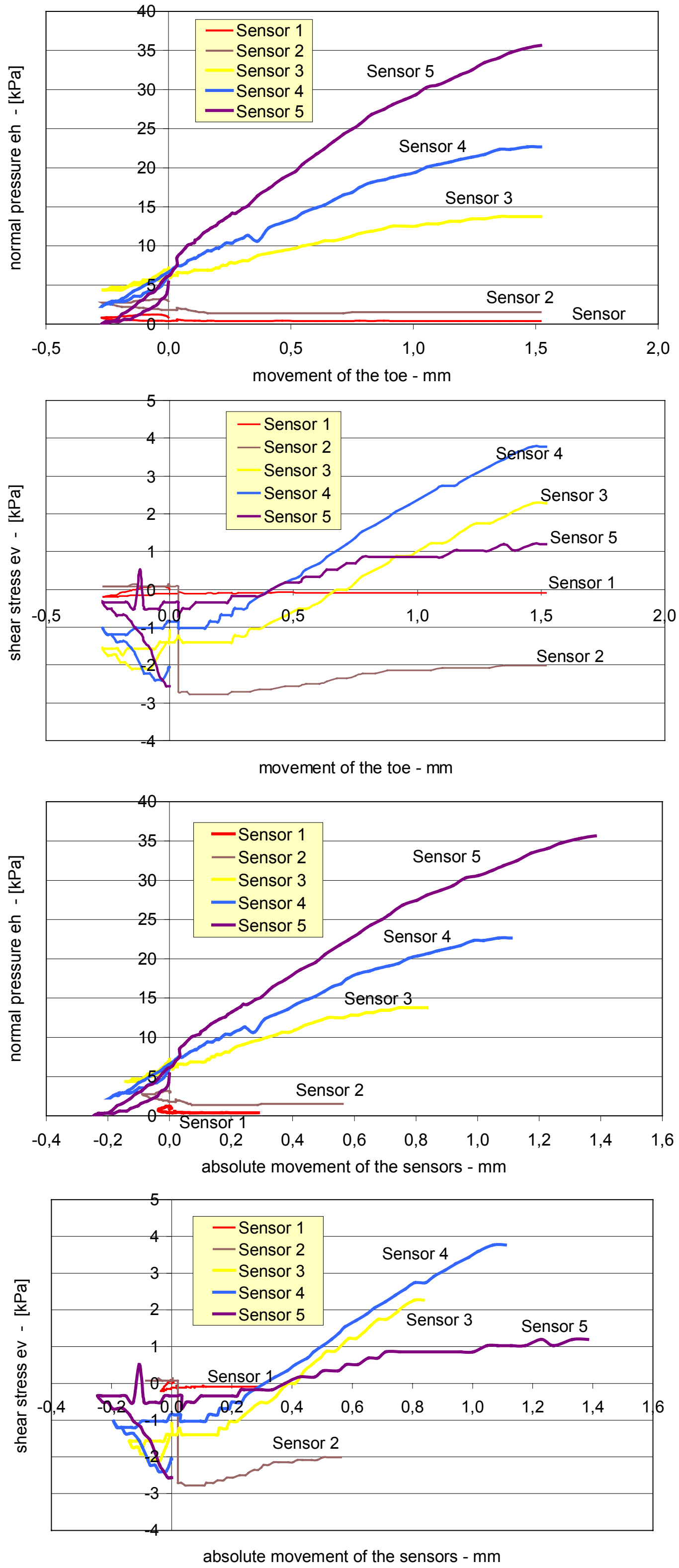

Fig. 16. Dependence of the both components of lateral pressure at rest (active and passive rotation about the top) in phases E3/2-0 and E3/2-1:
a) normal component on the toe movement,
b) shear component on the toe movement,
c) normal component on the absolute movements of sensors,
d) shear component on the absolute movements of sensors.

*h*istory and behaviour of both components of lateral pressure is obvious and unfitness of the upper dependences on the toe movement as well.

Fig. 17 shows behaviour of both lateral pressure components during the first phase of the experiment, i.e. *active* pressure at rest E3/2-0, and major passive movements at the first part of rotation about the top E3/2-1. The different *h*istory and behaviour of both components of lateral pressure is obvious but not so different as in the area pressure at rest. Unfitness of the upper dependences on the toe movement can be observed as well.

CONCLUSIONS

An analysis and numerical modelling are not the object of the paper. A number of other diagrams, results of visual monitoring of displacements into the mass, more detailed information, time instability are out of range the paper. Despite these limits can be stated:

a) The results have confirmed the different behaviour of the normal and shear components of lateral pressure in the range of *passive* pressure as well as in the range of *active* pressure.
b) The theoretical base of GLPT is not in discordance with new knowledge.
c) The new knowledge is useful to put the developed theory more precisely.
d) The monitored time instability extends acting of the general values pressure and the theory into the very important time-space.
e) Some contemporary knowledge leads to the conclusion that the natural state of granular mass is the state at rest and the mass appears the tendency to get into it.
f) This fact would lead to the very important statement that the natural values of lateral pressure are of the interval at rest and the mass appears tendency to get its lateral pressure into this interval.

These conclusions should be more verify. The continuing of the research is necessary.

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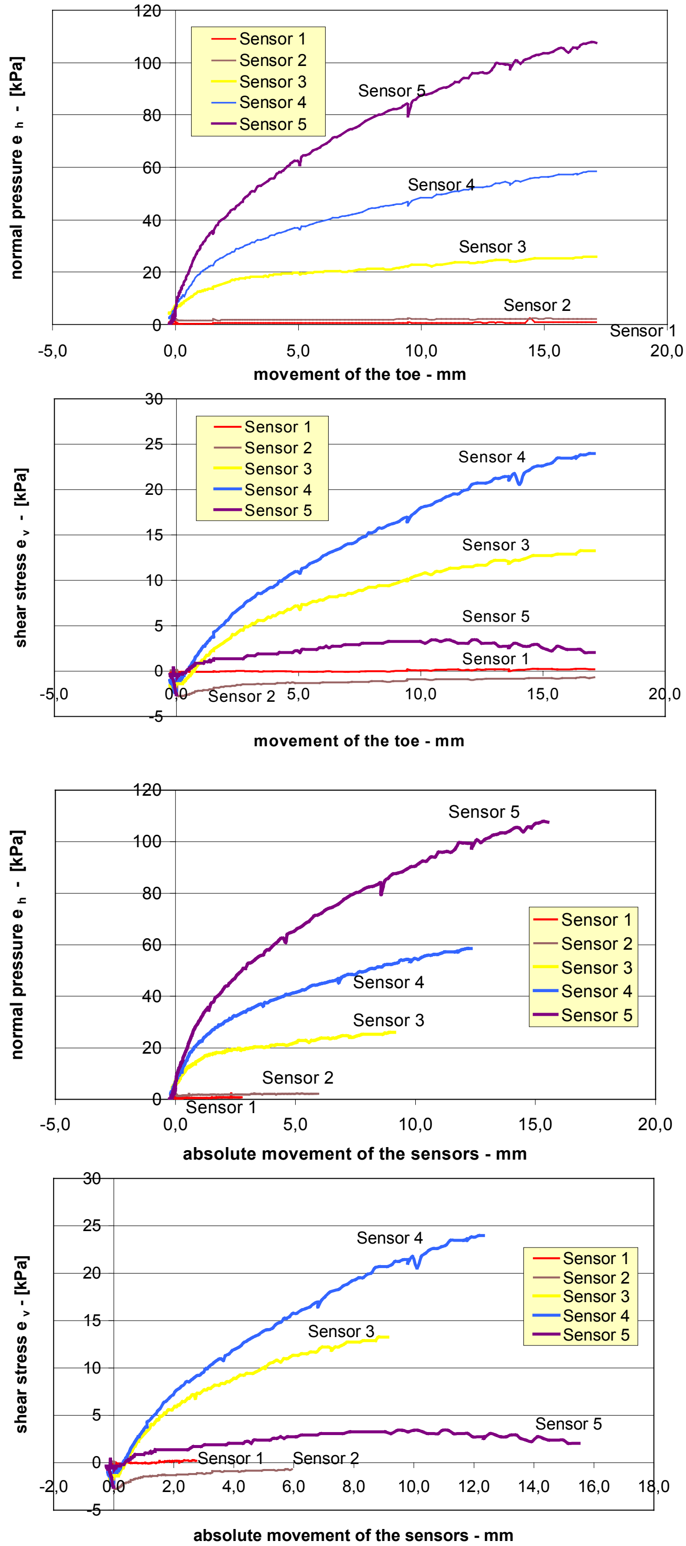

Fig. 17. Dependence of the both components of lateral pressure at rest (active and passive rotation about the top) in phases E3/2-0 and E3/2-1:
a) normal component on the toe movement,
b) shear component on the toe movement,
b) normal component on the absolute movements of sensors,
d) shear component on the absolute movements of sensors.

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