徐志英岩土工程论文选集

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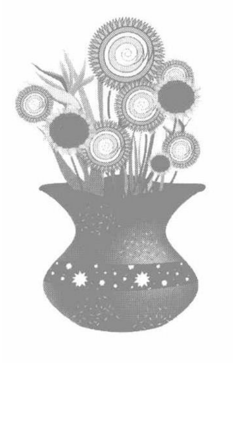
谨以此书献给徐志英教授
并祝贺他八十寿辰



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前 言

五月的江南，万象更新，百花争艳。值此之际，我们隆重庆祝我校资深教授、博士生导师、我国岩土工程界的老前辈徐志英教授八十华诞，特此向徐志英教授表示最热烈的祝贺和最诚挚的敬意。

徐志英教授是教育界的老前辈，为我国的教育事业和工程建设做出了巨大的贡献。徐志英教授于1924年5月生于江苏宜兴，1950年毕业于南京大学，从事教育事业越半个世纪，在教师岗位上辛勤劳动，在长期的高校教学和科研工作中，坚持倡导以理论、室内外测试和工程实践相结合的指导思想，为我国岩土工程的理论和实践做出了大量的贡献，造就和培养了一大批岩土工程专业骨干力量和高级人才，可谓桃李满天下。徐志英教授是我国著名的岩土工程专家、学者、博士生导师，在国际岩土工程界也有较高的声誉。徐志英教授学贯中西，成果卓著，主编《岩石力学》等教材3部，参编《土工原理与计算》等专著2部，译著《理论土力学》等10余部，约300余万字，还参编《中国大百科全书》、《中国农业百科全书》、《中国水利百科全书》和《水利词典》，参译《苏联建筑百科全书》等。在国内外发表论文90余篇。完成包括三峡工程重点课题2项在内的科研项目30多项。获国家级教学成果奖(2等奖)1次，水利部优秀教材奖(1等奖)2次，省部级科技进步1等奖、2等奖、3等奖各2次。

徐志英教授虽然已经退休，但他仍学术常青，继续关心学校的发展和岩土工程学科的发展，还不断地对青年教师进行指导，同时也相信年轻的一代一定能够把工作干得更好。对于徐志英教授为我国的教育和建设事业及河海大学的发展所付出的辛勤劳动和做出的突出贡献，我们再次表示深切的敬意和谢意。

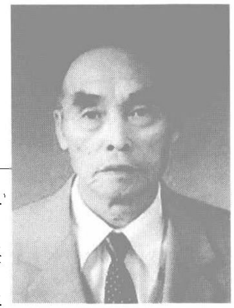
值此徐志英教授八十寿辰之际，我们谨从他发表的90多篇学术论文中，遴选出26篇具代表性的论文，汇编成《徐志英岩土工程论文选集》，以此表达我们对徐志英教授的衷心祝贺和敬意。

春风化雨，桃李芳菲，我们衷心祝愿徐志英教授健康长寿。

河海大学岩土工程科学研究所

2004年5月11日

徐志英教授简历



徐志英教授，1924年5月生，江苏宜兴人。1950年7月毕业于南京大学水利工程系。历任南京大学助教、华东水利学院（现河海大学）助教、讲师、副教授和教授，1986年经国务院批准为岩土工程专业博士生导师。1985年起曾兼任中国岩石力学与工程学会第一届、第二届理事，江苏省岩石力学与工程学会第一届、第二届副理事长，南京地基基础专业学会顾问，冶金部勘察科学技术研究所技术顾问，中国水利学会、中国力学学会以及中国地震学会会员。

徐志英教授从事高校教学科研四十余年，1985年我国首届教师节之际获江苏省优秀教育工作者称号，被授予奖状和奖章。1990年获国家教委表彰，被授予“从事高校科技工作四十年成绩显著荣誉证书”。1992年起享受国务院颁发的政府特殊津贴。1998年获严恺教育奖2等奖。徐志英教授于1952年协助黄文熙院士创建华东水利学院土力学教研室（现为河海大学岩土工程研究所），他是教研室最早四名成员之一，主编教材3部，参编专著2部，译著10余部，还参编《中国大百科全书》、《中国农业百科全书》、《中国水利百科全书》和《水利词典》，参译《苏联建筑百科全书》等。徐志英教授较早重视洋为中用，积极传播国外经验，翻译了英俄土力学书10余部（共300万余字），其中包括多部世界权威著作。例如，太沙基（“土力学之父”）著的《理论土力学》，苏联院士索柯洛夫斯基著的《松散介质静力学》、苏联院士弗洛林著的《土力学原理》等。徐志英教授于20世纪70年代初根据国家葛洲坝等水电工程建设的需要，又开始转入岩石力学学科的建设，于1973年首次开设岩石力学课程，是国内水利水电类专业开设最早的。他主编的《岩石力学》教材于1981年首次出版，1985年及1993年重新修订再版，该教材是国内水利水电类专业最早出版的岩石力学教材，迄今还被不少高校采用。该书于1995年获水利部第三届全国高校水利水电类专业优秀教材一等奖，1997年再获国家级教学成果二等奖。徐志英教授一贯提倡对于土力学、岩石力学（或岩土工程）这种应用学科在人才培养、科研和生产上都应以理论、室内外测试和工程实践密切结合作为指导思想，逐步使之形成教学、科研和生产密切结合的基地。1988年，我校岩土工程学科被批准为国家重点学科（国内仅2个）。徐志英教授为国家培养了包括十多名博士和硕士在内的一大批优秀人才。

徐志英教授积极响应党在1956年提出的“向科学进军”的号召，于1957年研制了

土的侧压力仪，并发表论文于《中国科学》上，常被国外杂志引用。该仪器已应用于太浦闸等的土工试验，当时即被水利电力部的《土工试验操作规程》所采用。1976 年唐山地震以后，他又在土石坝和尾矿坝的抗震方面做了大量的研究工作，在国内首先研制了振动单剪仪，提出了土石坝和尾矿坝的考虑孔压增长、消散、扩散的排水有效应力动力分析方法，并且将该方法编入研究生教材。徐志英教授在抗震方面完成了诸多科研和生产任务，例如岳城水库土坝、密云水库白沙土坝、石门水库、德兴铜矿尾矿坝、铜陵铜矿多个尾矿坝、马钢凹山尾矿坝和太钢峨口尾矿坝等的抗震研究，取得了显著的经济效益和社会效益。1991 年夏天，在洪水危及南京之际，徐志英教授亲临抗洪前线，为高淳胜利圩决堤及南钢山体滑坡等工程的抗洪抢险和修复加固工作出谋划策，做出了重要贡献，受到了江苏省政府在抗洪表彰会上的隆重表彰，为学校赢得了荣誉。

徐志英教授完成了包括三峡工程重点课题 2 项在内的科研项目 30 多项。主要科研成果有："土坝和尾矿坝的二维和三维有效应力动力分析"获国家教委 1986 年科技进步二等奖；"饱和砂基地震孔压扩散与消散计算"获水电部 1988 年科技进步一等奖；"土坝抗震分析"获水利部 1992 年科技进步三等奖；"非均质非线性各向异性岩土体强度理论与稳定分析"获水利部 1994 年科技进步一等奖；"各向异性非线性非均质材料极限分析理论及应用"获江苏教委科技进步三等奖；"交叉裂隙水流实验及渗流应力耦合分析方法研究"获电力部 1998 年科技进步理论成果二等奖。

徐志英教授在国内外重要刊物和学术会议论文集上发表了具有较高学术水平的论文 90 余篇。他曾多次参加国际会议和在国内讲学。1986 年，他出席在北京举行的复杂岩石的建筑物国际学术讨论会，1988 年赴日本东京、京都出席第九届世界地震工程会议，均在会上宣读论文。他在国际会议论文集上发表论文近 20 篇，在第九届、第十届、第十一届、第十二届世界地震工程会议论文集中均收录有他的论文。

徐志英教授的事迹已被数十种名人录所刊载，包括《江苏省高等学校教授录》、《中国当代自然科学人物总传》、《中国当代学者大辞典》、《世界名人录》、《世界优秀专家人才名典》、《世界华人专家名典》、《共和国专家成就博览》、《中国人才辞典》、《中国世纪专家》和《神州人物》等。

目录 Contents

A New Apparatus for the Determination of the Coefficient of Lateral Earth Pressure at Rest

Xu Zhiying(徐志英)

(Hydraulic Engineering College of Eastern China)

Introduction

In engineering design, the value of the coefficient of lateral earth pressure at rest is frequently required in the calculation of earth pressure on retaining structures or of Poisson's ratio in stress calculations and settlement estimations[1]. Therefore, it is practically important to determine the accurate values of these coefficients for different soils.

Various methods that have been used since 1920 for the determination of the coefficient of lateral earth pressure at rest are described in the first part of this paper. In the second part the author introduces a new apparatus for the same purpose which is evidently more preferable than those described above, from the viewpoints of both accuracy and efficiency.

The values of the coefficient of lateral earth pressure at rest for cohesive and sandy soil are given and their relations with void ratio, relative density, water content and degree of consolidation are fully discussed.

A general review of test data

Professor Terzaghi(1920) was the first one to carry out the experiment for determining the lateral earth pressure coefficient at rest[2]. The soil sample was put in a confined apparatus and vertical load was applied to the top of the sample as shown in Fig. 1. Two thin strips of metal were placed inside of soil consolidation devices. In one of the devices, one strip was placed horizontally and the other vertically with the soil sample. After the completed consolidation of soil, these strips were pulled out. The forces required for the horizontal and the vertical strips were measured and designated by p_h and p_v respectively, and thereby the coefficient of lateral earth pressure at rest is determined by the ratio $\xi = \frac{p_h}{p_v}$. Evidently, this method is suitable only for sand and remoulded clay[3]. For cohesive soils, the proportionate relation of ξ is not so simple as indicated above.

During the Second International Conference on Soil Mechanics and Foundation Engineering (1948), different results obtained by various investigators about this subject were presented and discussed, in which Terzaghi's result was quoted as follows[4]:

dense sand	$\xi = 0.4 \sim 0.5$
loose sand	$\xi = 0.45 \sim 0.5$
clay	$\xi = 0.6 \sim 0.75$

注：First published in Chinese in The Journal of Chinese Society of Civil Engineering, 1957, 4(2):197~208

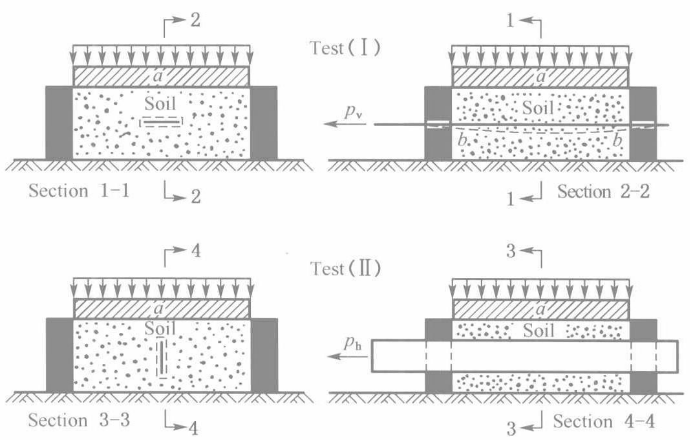


Fig. 1 Set-up used by Terzaghi for the determination of the at-rest coefficient of lateral earth pressure

Also in his book[5], Terzaghi claimed that the coefficient of lateral earth pressure at rest of sand depends upon the relative density and the process by which the deposit was formed. He suggested that the value could be obtained as high as 0.8 for sand compacted layer by layer, and equal to 0.4 and 0.5 for loose and dense sand respectively.

Professor Tschebotarioff[3] designed a much complicated apparatus and the testing procedure was rather tedious. According to his investigations, he pointed out that the data obtained by Terzaghi might be too high for clays and too low for sand in some cases.

Tschebotarioff also pointed out the influence of organic content on ξ. The higher the organic content, the lower the value of ξ, and ξ ranges from 0.37 to 0.24 for peat, due to its fibrous character. From DeBeer's test data, David Welch[6] computed the ξ values for 14 clay samples without more than 3.6 per cent organic content for consolidated equilibrium conditions. The variation of ξ was from 0.4 to 0.65 with 10 samples having values equal to 0.49, 0.50, 0.51 or 0.52.

To measure the lateral deformation of soil samples is the main feature of Professor Masloff's apparatus (USSR)[8]. According to the lateral deformation measured, Poisson's ratio μ and the coefficient of lateral earth pressure ξ can be computed. This device is only used for clayey soil in plastic and solid consistency. His test data were:

solid clay	$\xi = 0.11 \sim 0.25$
dense clay	$\xi = 0.33 \sim 0.45$
sandy clay	$\xi = 0.49 \sim 0.59$
plastic clay	$\xi = 0.61 \sim 0.82$

Professors Yaropolsky and B. D. Hvorslev both pointed out in their recent textbooks[9,10] the complexity involved in determination of the coefficient of lateral earth pressure. The former said that a further research of this problem would be rather limited, and suggested the following approximate values for practical uses:

$\xi = 0.7 \sim 0.75$	for clay
$\xi = 0.5 \sim 0.7$	for sandy clay
$\xi = 0.4$	for sand

He also said that ξ can be reduced to 0.35 for coarse sand in loose state and this value increases as relative density increases.

According to the values of Poisson's ratio given by Hvorslev[10], the author computed the ξ values as follows:

$\xi = 0.17$	for gravel
$\xi = 0.25$	for sand
$\xi = 0.33$	for sandy loam
$\xi = 0.43 \sim 0.54$	for sandy clay
$\xi = 0.54$	for clayey soils
$\xi = 0.66$	for clay

The following is some information on this subject from German literature.

Firstly, Bernatzik[11] discovered that the coefficient of lateral earth pressure depends on its void ratio ε.

$\xi = 0.49$	for $\varepsilon = 0.6$
$\xi = 0.52$	for $\varepsilon = 0.7$
$\xi = 0.64$	for $\varepsilon = 0.88$

Secondly, Frieser[11] obtained the following divergent values for sands of different grain shapes:

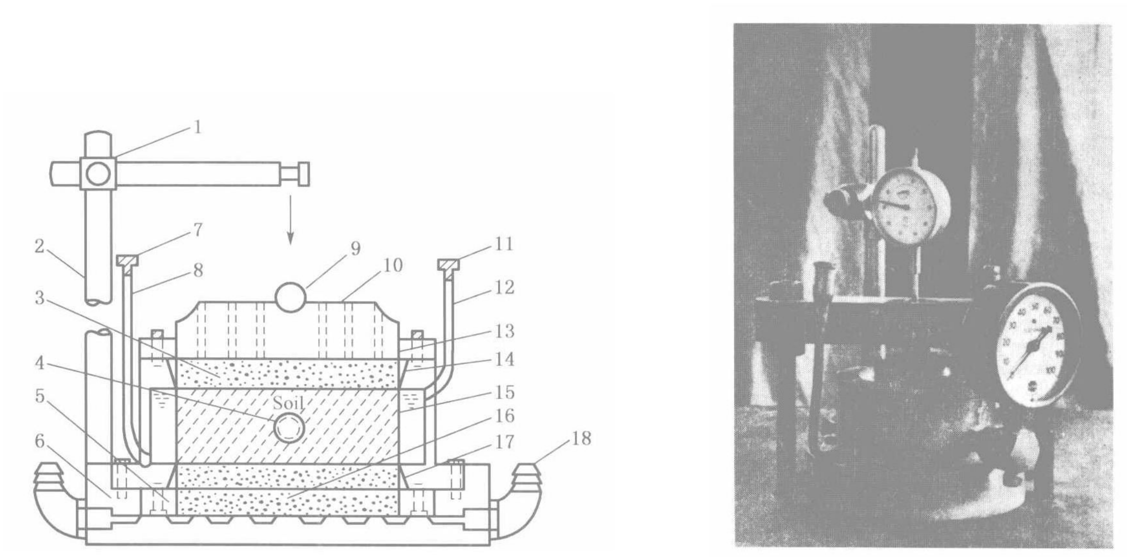
$\xi = 0.50$	for spherical sand
$\xi = 0.42$	for flaky sand

A table for the coefficient of lateral earth pressure ξ for non-cohesive soil was given by Schultze-Muhs[11]. It ranges from 0.14(theoretical lower limit) to 0.65(the highest obtained by experiment). Fedorow and Malysehew gave $\xi = 0.18 \sim 0.52$ for non-cohesive soils, and they claimed that the ξ-value only depends on initial void ratio[11].

Jänke, Martin and Plehm measured the coefficient of lateral earth pressure by means of tri-axial compression machine[11]. Detailed descriptions were also given in their paper. The testing procedures are similar to those in tri-axial shear test, except that a special device is used in order to prevent the soil sample from lateral swelling. The soil sample is put in the rubber membrane. The pressure chamber is then filled with fluid(water). The increase of the lateral pressure due to the vertical load from the piston is transmitted by the fluid and measured with a Bourdon gauge. The correlation between the coefficient of lateral earth pressure and the initial void ratio ε_a obtained is also given. One set of these results is quoted here:

$\xi = 0.23$	for $\varepsilon_a = 0.49$
$\xi = 0.29$	for $\varepsilon_a = 0.59$
$\xi = 0.34$	for $\varepsilon_a = 0.71$

As mentioned above, the results of investigation seem to be quite divergent. This was also pointed out during the Second International Conference on Soil Mechanics and Foundation Engineering held in Rotterdam early in 1948. Therefore it is urgent to have a new device for further research. At the beginning, the author used a stabilometer (usually used for testing bituminous materials by road engineer) for the purpose of determining the coefficient of earth pressure at rest. This apparatus is obviously deficient as compared to the new one which the author used in later stage. In addition to the determination of the coefficient of lateral earth pressure, the new apparatus can also be used to determine compression and swelling curves, especially the correlation between void ratio ε and $\sigma_x + \sigma_y + \sigma_z$, which is necessary in the close estimation of foundation settlement[1].



(a) (b)

Fig. 2 Apparatus for measuring lateral earth pressure

1, 2—dial gauge holder 3, 16—porous stone 4—hole for Bourdon gauge 5, 13—cap ring 6 base 7, 11- plug 8 water inlet 9- steel ball 10—piston 12—water outlet 14—wedge ring 15—rubber membrane 17—water ring 18—drain

Details of new apparatus and the testing procedures

1. The main parts of the apparatus.

The apparatus is similar to the ordinary consolidation device, except that there is a "water ring" for enclosing soil sample instead of the common solid copper ring. The water ring consists of a C-shaped wall section which is covered by rubber membrane to form a closed hollow space. Then water is filled into this space. In the upper and lower corner, small holes are provided on the wall for water inlet and outlet(also used for expelling air) tube connections. In the middle of the wall there is a hole for Bourdon gauge which is used in this device for measuring lateral pressure. The rubber membrane is held tight by screwing on the wedge ring and the cap ring. After that the water ring is put upon the base and fixed by screws. The soil sample is placed in the space enclosed by the membrane, and porous stone is fixed both on the top and bottom for drainage. After the piston and steel ball are put on, the whole set-up is placed upon the loading platform of an ordinary consolidation device for applying vertical load.

Under a certain load, the value of lateral earth pressure can be directly read from the Bourdon gauge, while the compression of the soil sample is obtained from the dial gauge readings. All the tubings and connections of the water ring should be watertight, and in order to reduce the effect of wall(membrane) friction, the height of sample as compared with its diameter should not be too great.

The details and appearance of the new device are shown in Fig. 2(a) and Fig. 2(b).

2. Preparation of samples and test procedures.

The rubber membrane is first fixed in position by the wedge ring and cap ring. The thickness of the membrane used is about 0.4 mm and thick membrane is unsuitable. Water is introduced through the inlet tube until the whole water ring is filled, and water begins to overflow from the outlet tube. After the outflow water no longer carries away any air bubble for 2~3 minutes, both the inlet and outlet tubes are plugged tight. The membrane used should be fixed tight and straight, and of light colour in order to detect any concealed air bubble in the water ring. If the membrane is thin, such bubbles can be easily detected and watering is renewed until there is no more bubbles. However, too thin a membrane will often induce breakage of membrane during the test. A short length of brass tube with its outside diameter equal to that of the sample is temporarily pushed into the water ring with the aim to keep the membrane straight when water is filling into the ring. When all these have been done, the water ring is fixed onto the base and the soil sample is placed or pushed in.

The height and the diameter of the soil sample are three cm and eight cm respectively. Sand sample is directly put into the device and compacted to different densities. Its corresponding initial void ratio can be easily computed.

For soils in liquid consistency which are unable to stand, samples can also be directly put into the device. Both initial water contents and void ratios should be determined.

For soils of plastic and solid consistency disturbed and undisturbed, the rubber membrane is first oiled and then the sample is trimmed and pushed into the cylinder. The water content and void ratio should be determined before test for every sample.

After vertical load is added, lateral pressure is read from the Bourdon gauge. If the lateral pressure coefficient after consolidation or the relation between $\sigma_x+\sigma_y+\sigma_z$ and ε is required for clay, we can also observe the variation of lateral pressure under a constant vertical load during the test.

In order to prevent the sample from drying, water box outside the apparatus is used for saturated clay, and moistened cotton is used for clay with natural water content.

The scale of the Bourdon gauge used is divided to 1 lbs/in^2 and can be estimated to 0.1 lbs/in^2.

3. Special precautions:

(1) No leakage is allowed in the entire water circulation system, so absence of small sand holes in the ring wall is important. Otherwise the wall will "perspire" and water will percolate through the wall.

(2) The friction resistance between sample and rubber membrane affects the transmission of vertical load, so the thickness of sample should be considerably smaller than its diameter.

(3) Any air bubble in the water ring may reduce the lateral pressure readings. One must be sure that there is no air bubble in the water ring, and it is better to use boiled water (after cooling).

(4) As the compression of the membrane itself also affects the gauge readings, the rubber membrane should be made as thin as possible (0.4 mm is preferred).

(5) The bourdon gauge should be calibrated before use.

(6) Instead of two porous stones, two porous copper plates may be used in order to reduce the friction force between stone faces and soil sample to minimum.

Experimental results

Two disturbed samples (one is sand and the other is sandy clay from the campus of our college) are used. The physical properties of sand are: specific gravity 2.66, max., void ratio $\varepsilon_{max}=0.99$, min., void ratio $\varepsilon_{min}=0.49$. As to its grain-size distribution the sand used is classified as coarse sand (according to the standards for civil and industrial building foundations adopted by USSR in 1955). Most of its particles are spherical and with moderate coarseness. During the test, the samples are in dry state and compacted to different densities. The vertical load intensity varies from 1~4 kg/cm^2 which is usually encountered in engineering practice and the initial void ratio for each sample is determined. For each sample, the increments of vertical load p_v are: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, and 4.0 kg/cm^2; so the author obtain the corresponding readings of lateral pressure p_h. A summary of the results is presented below:

(1) From the p_v—p_h relation diagram (Fig. 3), the author find that the loading curve is a fairly straight line and passes through the origin, while the unloading curve does not coincide with the loading curve and possesses strong curvature, thereby forming a hysterisis loop. The slope of the tangent to the unloading curve increases as the vertical load decreases.

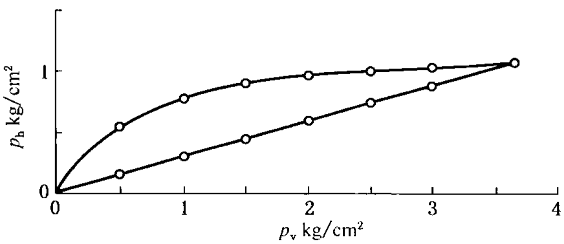


Fig. 3 Relations between p_v and p_h for sand

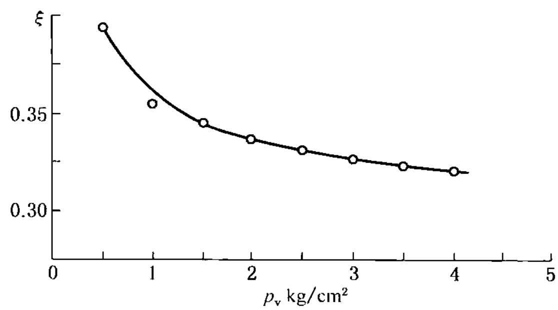


Fig. 4 Relation between lateral earth pressure coefficient ξ and vertical pressure p_v for sand

(2) As the loading curve is a straight line, ξ-values seem to be independent of the magnitude of vertical load. Yet by calculating the ratio between every horizontal pressure and its corresponding vertical pressure, it is found that the ξ-values decrease slightly as p_v increases, and this is tabulated below and plotted in Fig. 4.

$p_v = 0.5;\ 1.0;\ 1.5;\ 2.0;\ 2.5;\ 3.0;\ 3.5$

$\xi = 0.398;\ 0.351;\ 0.349;\ 0.334;\ 0.329;\ 0.326;\ 0.317$

(3) Generally speaking, the ξ-values depend upon the initial void ratio. The larger the initial void ratio, the higher the ξ-values. In other words, the smaller the relative density of sand, the higher the ξ-values. The test results of four sand samples are shown in Fig. 5.

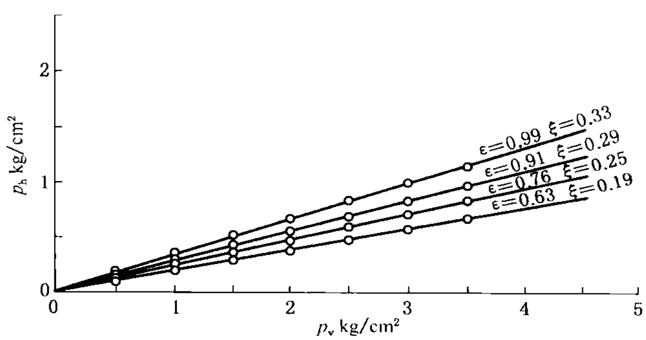


Fig. 5 Relations between p_v and p_h for sand with different initial void ratios

(4) All of the ξ-values with their corresponding values of void ratio or relative density D are shown in Fig. 6 and

Fig. 7 respectively.

(5) The ξ-value of the coarse sand tested varies from 0.33 to 0.19, and its Poisson's ratio from 0.25 to 0.16.

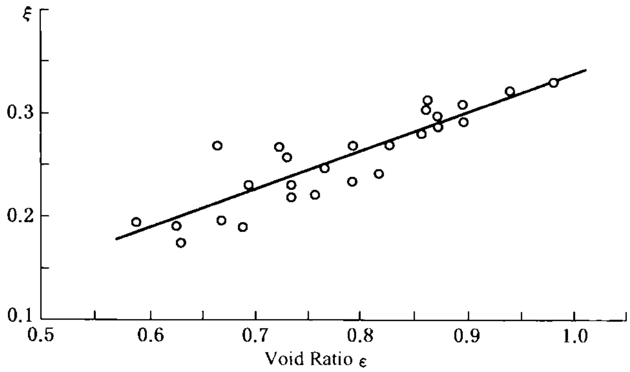


Fig. 6 Relation between lateral earth pressure coefficient and initial void ratio for sand

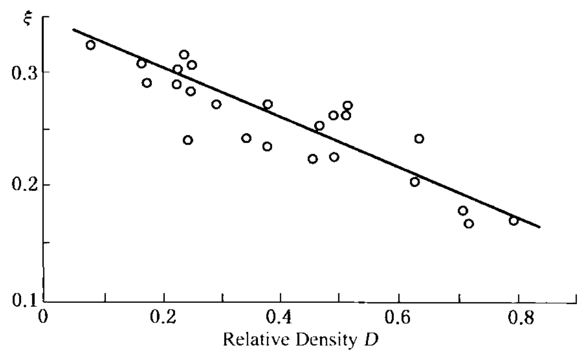


Fig. 7 Relation between lateral earth pressure coefficient and relative density D for sand

The physical properties of the sandy clay used are: liquid limit = 37%, plastic limit = 20.8%, shrinkage limit = 11.1%, specific gravity = 2.70, and its grain-size distribution is:

2 ~ 0.05 mm	9%
0.05 ~ 0.002 mm	74%
< 0.002 mm	17%

During the test, different water contents of this sample were used under both consolidated and unconsolidated conditions. Here are the test results of sandy clay:

(1) After vertical load is applied, the horizontal pressure varies with time. At the beginning, the latter increases a little, and then decreases gradually until a certain constant value is reached.

(2) Fig. 8 shows the relationship between the initial horizontal pressure and its corresponding vertical pressure under the unconsolidated condition.

The loading curve is also a straight line, usually not passing through the origin (except for samples with very high water content). The unloading curve does not coincide with the loading curve, but sometimes intersects with the ordinate axis, somewhat like that of sand but not so remarkable.

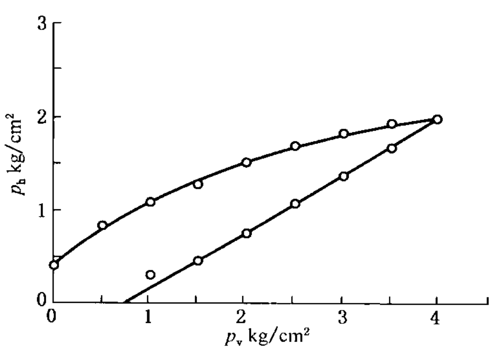


Fig. 8 Relation between p_v and p_h for sandy clay

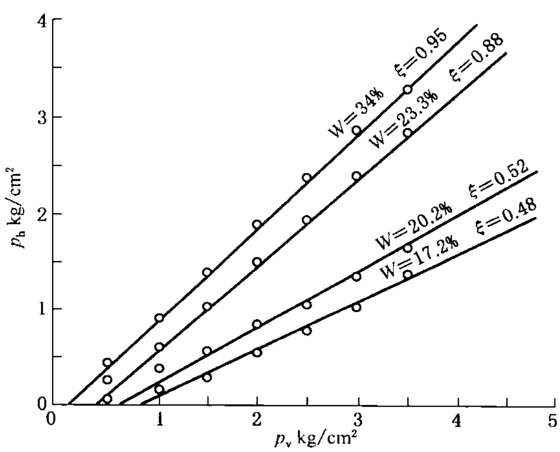


Fig. 9 Relations between p_v and p_h for sandy clay with different water contents

At the same time the author finds that the coefficient of lateral pressure at rest of sandy clay depends mainly upon its water content as shown in Fig. 9.

If the ξ-values are plotted against their corresponding water contents, it turns to Fig. 10. From this figure, it is evident that the coefficient of lateral earth pressure increases rapidly as the water content increases. As soon as the water content of sample approaches its liquid limit, the ξ-value is approximately equal to unity.

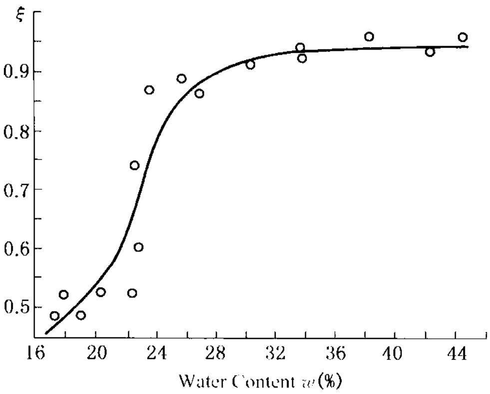


Fig. 10 Relation between ξ value and water content for remoulded sandy clay (unconsolidated)

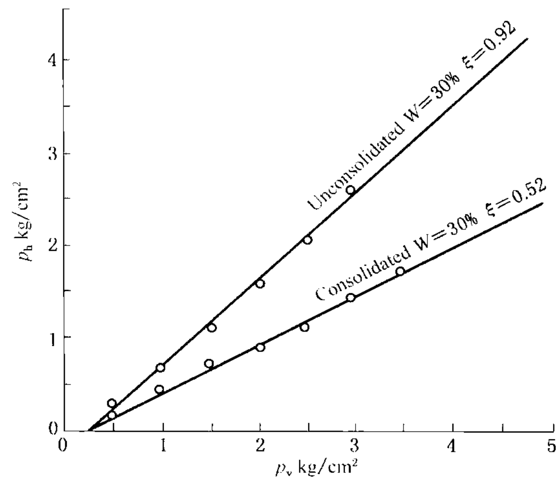


Fig. 11 Relations between p_v and p_h for sandy clay with the same initial water content of 30%

(3) Under the consolidated condition (after 24 hours), there is also a straight line relationship existing between the horizontal pressure and its corresponding vertical pressure, and the straight line intersects with the abscissa axis. The ξ-value obtained by this method is much smaller than that obtained under the unconsolidated condition, but with the same initial water content. Test results for samples of 30% water content under these two different conditions are shown in Fig. 11.

Conclusions

(1) The coefficient of lateral earth pressure not only depends upon the kind of soils but also upon its void ratio, relative density (for sands), water content, loading and consolidation conditions (in the case of cohesive soils). The coefficient varies within wide limits, so it is necessary to determine the coefficient in each case under definite conditions in accordance with the practical environment.

(2) Terzaghi's values ($\xi = 0.4$ for loose sand, 0.5 for dense sand, and 0.8 for compacted sand) and Yaropolsky's opinion regarding the coefficient of lateral pressure of sand (i.e. values of ξ increase as relative density increases) are *entirely contradictory* to those data obtained by Bernatzik, Jänke and the author. Theoretically, the at-rest coefficient should be a little larger than the coefficient of active earth pressure K_A. ξ-values obtained by the author's experiment are in agreement with this conclusion. The values of K_A for the sand used at void ratios $\varepsilon = 0.6$ and 0.7 (computed from angle of internal friction φ which is obtained by box-shear test), and the corresponding values of ξ are tabulated below.

(3) The ξ-values of sand obtained by the author's apparatus are very close to those obtained by Hvorslev in USSR and Jänke in Germany.

(4) The ξ-values of consolidated sandy clay obtained by the author's apparatus are nearly equal to those suggested by Hvorslev.