Three-dimensional poro-elasto-plastic model for wave-induced seabed response around submarine pipeline

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ARTICLE INFO

Article history:

Received 25 July 2014

Received in revised form

23 October 2014

Accepted 6 November 2014

Available online 26 November 2014

Keywords:

Three-dimensional model

Seabed response

Poro-elasto-plastic

Submarine pipeline

ABSTRACT

The evaluation of the wave-induced excess pore pressure around a buried pipeline is particularly important for pipeline engineers involved in the design of offshore pipelines. Existing models for the wave-induced seabed response around submarine pipeline have been limited to poro-elastic soil behavior and de-coupled oscillatory and residual mechanisms for the rise in excess pore water pressure. To overcome the shortcoming of the existing models, in this study a three-dimensional poro-elasto-plastic soil model with submarine pipeline is established, in which both oscillatory and residual mechanisms can be simulated simultaneously. With the proposed model, a parametric study is conducted to investigate the relative differences of the predictions of the wave-induced pore pressure with poro-elasto-plastic model. Based on numerical examples, it can be concluded that the poro-elasto-plastic behaviors of soil have more significant influence on wave-induced pore pressure of seabed around submarine pipeline. As the seabed depth increases, the normalized pore pressures decrease rapidly at the upper part of seabed, and then change slightly at the lower part of the seabed. Soil permeability and wave period have obvious influence on the wave-induced normalized pore pressure.

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1. Introduction

Wave-induced seabed instability is one of the key factors that must be considered in the design of foundation in the vicinity of marine structure such as pipelines and breakwaters. It has been well documented that ocean waves propagating over the ocean surface exert dynamic pressure fluctuations on the sea floor. The dynamic pressure fluctuations further induced excess pore pressure within the soil skeleton, and then subsequently lead to the variation of stresses in seabed. This mechanism involved in this process is rather complicated since it is a coupled problem among soil, pore fluid and structures. Furthermore, it is vital because that it could lead to the reduction of strength or even liquefaction of the foundation around marine structures. It has also been reported that the soil supporting the pipeline may fail due to liquefaction, resulting in the self-burial of the entire pipeline [1,18]. Moreover, it has been observed that a deep scour hole exists near the tip of a marine structure [5]. Without proper maintenance at these sites, failure of structures may occur.

Generally speaking, two mechanisms of the wave-induced seabed response in the rise in pore water pressure have been observed in the field measurements and laboratory experiments, depending upon the way the excess pore pressure is generated [15,23]. One is caused by residual or progressive nature of the excess pore pressure, which appears at the initial stage of cyclic loading. This type of soil response is similar to that induced by earthquakes, which resulted in the build-up of the excess pore pressure. The other is generated by transient or oscillatory excess pore pressure that is accompanied by the damping of amplitude and phase lag in the pore pressure [22,7,8].

Numerous investigations for the wave-induced seabed response have been carried out since the 1970s. Among these, analytical approximations have been developed by various researchers such as Madsen [11], Yamamoto et al. [22], Mei and Foda [14], and Jeng [7,8]. Numerical simulations also have been widely applied to examine such a problem in recent years. Cheng and Liu [4] considered a buried pipeline in a region that is surrounded by two impermeable walls. Magda [13] considered a similar case with a wider range of degree of saturation. Luan et al. [10] considered soil–pipeline contact effects and inertial forces in a new model. At the same time, experimental work has attracted attention from researcher and pipeline engineers. Teh [19] considered the two recent contributions to the problem of the on-bottom stability of marine pipelines on liquefied seabed.

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brief outline of the theory. Further details can be found in Zienkiewicz et al. [25]

- (1) Equilibrium of mixture is written as (in tensorial form)

$$\sigma_{ij,j} + \rho b_i - \rho \ddot{u}_i - \rho_f (\dot{w}_i + w_j w_{ij}) = 0, \quad (7)$$

where σ_{ij} is the total stress tensor (tensile positive), $\ddot{u}_i$ and w_i are the acceleration of the solid skeleton and average (Darcy) fluid velocity respectively, b_i is the body acceleration per unit mass, ρ_s , ρ_f and ρ are the densities of the solid grain, fluid and mixture, respectively, with $\rho = (1-n)\rho_s + n\rho_f$ and n being the porosity of the porous media. The underlined terms represent the fluid acceleration relative to the solid and the convective terms of the acceleration.

- (2) Equilibrium of fluid

$$-p_{,i} - F_i - \rho_f \ddot{u}_i - \rho_f (\dot{w}_i + w_j w_{ij})/n + \rho_f b_i = 0 \quad (8)$$

where F represents the viscous drag forces, assuming the Darcy seepage law can be written as $k_{ij}F_j = w_i$, where k is the permeability of soil with the dimensions of [length]³[time]/[mass], which can be expressed by the usual soil mechanics convention k' , with $k = k'/\rho_f g'$, where ρ_f and g' are the fluid density and gravitational acceleration, respectively, at which the permeability is measured.

- (3) Conservation of mass of fluid phase:

$$w_{i,i} + \dot{\varepsilon}_{ii} + \frac{n\dot{p}}{K_f} + \frac{(1-n)\dot{p}}{K_s} - \frac{K_T}{K_s} \left(\dot{\varepsilon}_{ii} + \frac{\dot{p}}{K_s} \right) + n \frac{\dot{\rho}_f}{\rho_f} + \dot{s}_0 = 0 \quad (9)$$

where $w_{i,i}$ is the flow divergence in the unit volume, $\dot{\varepsilon}_{ii}$ is the increased volume due to a change in strain, $n\dot{p}/K_f$ is the additional volume stored by compression of void fluid due to the fluid pressure increase, $(1-n)\dot{p}/K_s$ is the additional volume stored by the compression of grains by the fluid pressure increase, $-K_T(\dot{\varepsilon}_{ii} + \dot{p}/K_s)/K_s$ is the change in volume of the solid phase due to a change in the inter-granular effective contact stress, and the underlined part $n\dot{\rho}_f/\rho_f + \dot{s}_0$ corresponds to a change of density and rate of volume expansion of the solid in the case of thermal change and is negligible in general.

The mass conservation equation can be further expressed by using the definition of α and Q

$$w_{i,i} + \alpha \dot{\varepsilon}_{ii} + \frac{\dot{p}}{Q} + n \frac{\dot{\rho}_f}{\rho_f} + \dot{s}_0 = 0 \quad (10)$$

where K_T is the average bulk modulus of the solid skeleton, K_s is the average material bulk modulus of the solid components of the skeleton and K_f is the bulk modulus of the fluid, with $1/Q \equiv n/K_f + (\alpha - n)/K_s \cong n/K_f + (1-n)/K_s$ and $\alpha = 1 - K_T/K_s$.

Coupling Eqs. (8) and (10), together with Eq. (7), neglecting the underlined terms which are apparently small under general earthquake analysis, the governing equation can be expressed as follows:

$$\sigma_{ij,j} + \rho b_i - \rho \ddot{u}_i = 0 \quad (11)$$

$$\left(k_{ij} \left(-p_{,j} - \rho_f \ddot{u}_j + \rho_f b_j \right) \right)_{,i} + \alpha \dot{\varepsilon}_{ii} + \frac{\dot{p}}{Q} = 0 \quad (12)$$

Due to this simplified equation set only containing two dependent variables u and p , it is usually called u - p approximation form.

3.2. Consolidation model

If we further ignore the acceleration due to the soil motion, the u - p approximation will become the consolidation formulation:

$$\sigma_{ij,j} + \rho b_i = 0 \quad (13)$$

$$\left(k_{ij} \left(-p_{,j} + \rho_f b_j \right) \right)_{,i} + \alpha \dot{\varepsilon}_{ii} + \frac{\dot{p}}{Q} = 0 \quad (14)$$

Eq. (14) is the conventional consolidation equation [2]. As reported in the previous work [9], the conventional consolidation is sufficient for most cases of the wave-induced soil response in marine environment. Therefore, the consolidation equation is used in this study.

3.3. Soil constitutive model

For elastic behavior, based on poro-elastic theory, the stress-strain relationship can be expressed as

$$\dot{\sigma}_{ij} = D_{ijkl}^e \dot{\varepsilon}_{kl} \quad (15)$$

where D_{ijkl}^e stands for elements of constitutive matrix that describes the constitutive stress-strain relation, which can be expressed as the following expression for the case of isotropic elasticity:

$$D_{ijkl}^e = \lambda \delta_{ij} \delta_{kl} + 2G \delta_{ik} \delta_{jl} \quad (16)$$

where λ is Lamé's constant and G is the shear modulus.

In (15), $\varepsilon^e = \varepsilon_{ij}^e = (u_{i,j} + u_{j,i})/2$ is the small strain tensor and u is the soil displacement.

The generalized plasticity model by Pastor and Zienkiewicz [17] is used in this study to describe the elasto-plastic soil behavior under wave loadings. This poro-elasto-plastic model used in this paper can simulate the pore water pressure under dynamic loading, including seismic loading and wave loading, which can describe residual mechanism of pore water pressure better. More details about Pastor and Zienkiewicz soil constitutive model can be available from Ou [16] or Zienkiewicz et al. [25].

3.4. Validations

First, a 3D linear elastic analysis on wave-induced soil response in finite thickness soil bed was performed and the numerical results were compared with the analytical solution [6]. All degrees of freedom at the two ends of the seabed are totally tied to form a repeated boundary condition appropriately reflecting the infinite lateral extent of the seabed. The design wave condition at North Sea is used as an example [22], i.e., water depth (d) is 10 m and the wave height (H) is 5 m. For the seabed, the shear modulus and Poisson's ratio are taken to be 10 MPa and 1/3 respectively. The porosity is 0.3 and the permeability is taken as 0.001 m/s. 1664 Lagrangian brick elements of 27-noded are used in the analysis, and the finite element mesh. An excellent agreement between the numerical results (solid symbols and lines) and the analytical solution (hollow symbols and lines) [6] is shown in Fig. 2.

Second, validation of the present model is to compare with a set of experimental data available in the literature [20]. The experimental study was conducted in the J. H. DeFree Hydraulic Laboratory at Cornell University. The input data of seabed characteristics and pipeline characteristics for the experiment and numerical calculations are listed in Tables 1 and 2, and Fig. 3. As Turcotte et al. [20] reported that the sediment was prepared to be uniform. In Fig. 3, the solid line represents the present model, “●” denotes the experimental data, and “Δ” is the results of Cheng and Liu. As seen in Fig. 3, the present model reasonably agrees with the experimental data, and slightly better than the results of Cheng and Liu. The two numerical results showed slight difference probably because two different numerical methods were applied to this test. Cheng and Liu performed that using a boundary Integral Equation Method, the Finite Element Method is adopted in this paper.

Fig. 2. Comparison between 3D numerical solution and the analytical solution of Hsu and Jeng.

Table 1

Input data of seabed characteristics for the comparison with the experimental data.

Seabed characteristics	
Seabed length (l) (m)	4.57
Seabed thickness (h) (m)	0.826
Modulus of deformation (E_s) (N/m ²)	1.7×10^6
Permeability (k_s) (m/s)	1.1×10^{-3}
Density (ρ_s) (kg/m ³)	1700
Poisson ratio (ν_s)	0.33
Porosity (n_s)	0.42
Degree of saturation (S_r)	0.95

Table 2

Input data of pipeline characteristics for the comparison with the experimental data.

Pipeline characteristics	
Buried depth of pipeline (b) (m)	0.083
Pipeline outer diameter (D) (m)	0.168
Pipeline thickness (t_p) (m)	0.01
Young's modulus (E_p) (N/m ²)	3.9×10^9
Density (ρ_p) (kg/m ³)	1400
Poisson ratio (ν_p)	0.35

Another validation of the proposed model for accumulated pore pressure is made with a set of experimental results in the literature [21]. The input data of seabed characteristics and pipeline characteristics for the experiments and numerical calculations are described as follows: the permeability of soil is 7.4×10^{-2} m/s and the shear modulus is 6.2×10^{-7} N/m²; the diameter of pipeline is 40 mm and the burial depth with 93 mm. The wave characteristic parameters are described as follows: the water depth is 0.4 m, the wave period is 2 s and wave height is 15 cm in the experiments. In Fig. 4, the solid line represents the experimental results and the dashed line represents the results of present numerical computation results. Fig. 4(a) shows the results of residual pore pressure with time at the top of buried pipeline and Fig. 4(b) shows the results of residual pore pressure with time at the bottom of buried pipeline. As seen in Fig. 4(a) and (b), the two present numerical results are closed to the experimental data. The results of numerical computation and experiments have the same trend. The numerical results showed slight difference in the latter part of wave loading, it may be because the

Fig. 3. Comparison with the experimental data and the numerical result of Cheng and Liu.

Fig. 4. (a) Comparison of accumulation pore pressure at the top of pipeline with the experimental result of Pan et al. (b) Comparison of accumulation pore pressure at the bottom of pipeline with the experimental result of Pan et al.

accumulated pore pressure dissipates more obviously in the experiments than in the numerical computation.

4. Results and discussions

One of contributions of this study is the consideration of elastoplastic soil behavior in the wave-induced seabed response. To illustrate the difference of the predicted wave-induced pore

Table 3
Input data of seabed characteristics for parametric study.

Seabed characteristics	
Seabed length (l) (m)	75
Seabed thickness (h) (m)	15
Seabed width (m) (m)	10
Modulus of deformation (E_s) (N/m ²)	7×10^7
Permeability (k_s) (m/s)	1.0×10^{-3}
Density (ρ_s) (kg/m ³)	1700
Poisson ratio (ν_s)	0.30
Porosity (n_s)	0.40
Degree of saturation (S_r)	1.0

Table 4
Input data of pipeline characteristics for parametric study.

Pipeline characteristics	
Buried depth of pipeline (b) (m)	1.0
Pipeline outer diameter (D) (m)	1.0
Young's modulus (E_p) (N/m ²)	3.0×10^{10}
Density (ρ_p) (kg/m ³)	2100
Poisson ratio (ν_p)	0.30

Table 5
Input data of wave characteristics for parametric study.

Wave characteristics	
Wave length (L) (m)	23.2
Wave period (T) (s)	7.5
Wave height (H) (m)	1.0
Water depth (d) (m)	1.0

Table 6
Input data of parameters for PZ3 model for parametric study.

Parameters for PZ3 model	
K_{evo} (kPa)	770
G_{eso} (kPa)	1155
p'_0 (kPa)	4
M_g	1.15
M_f	1.035
α_f	0.45
α_g	0.45
β_0	4.2
β_1	0.2
H_0	600
H_{U0} (kPa)	40,000
γ_u	2.0
γ_{DM}	4.0

pressure between the previous elastic and the present elasto-plastic models, a parametric study is performed. The influence of wave period, soil permeability and shear modulus and elasto-plastic parameters is examined. The input data of seabed characteristic, pipeline characteristic and wave characteristic for numerical examples in parameters study are tabulated in Tables 3–5.

The Pastor Zienkiewicz mark III elasto-plastic Model (PZ3 Model) was adopted in the analysis. The elasto-plastic parameters presented in Table 6 are given by Zienkiewicz et al. [25] to simulate the dense seabed. It should be noted that K_{evo} in Table 6 represents the value of Bulk modulus of soil at the reference mean effective stress p'_0 , and

Fig. 5. (a) Three-dimensional mesh of wave-seabed-pipeline interaction problem. (b) The cross-section sketch of Fig. 5(a). (c) Three-dimensional scenography sketch of Fig. 5(a).

G_{eso} represents the value of three times of shear modulus at the reference mean effective stress (G'_{eso} represents the shear modulus in the following). The elastic parameters of the linear elastic analysis are calculated based on the value of K_{evo} and G_{eso} with the reference mean effective stress at the middle depth of the seabed.

Fig. 5(a) shows the finite element mesh of 3D wave-seabed-pipeline interaction problem model. Fig. 5(b) shows the cross-section sketch of Fig. 5(a). Fig. 5(c) shows the three-dimensional scenography sketch of Fig. 5(a). Fig. 5(a), (b) and (c) show all the sketch of the same seabed model with submarine pipeline. As concentration of stresses is expected, local refinement of the finite element mesh always has taken into account in the region near the pipeline. To improve the accuracy of the solution, two different mesh systems are used for the model. As shown in Fig. 6, this kind of mesh has been used for tackling the problem around a pipe-like structure [12].

4.1. The distributions of pore pressure along seabed depth

In order to examine the distributions of normalized pore pressure versus soil depth, Fig. 7 shows the results of numerical computation with the proposed 3D pore-elasto-plastic model. In Fig. 7, the normalized pore pressure means the ratio of p to p_0 , p denotes the excess pore water pressure obtained by numerical computation and p_0 denotes the amplitude of the wave pressure at the surface of the seabed. The solid line and symbols denote the results of normalized pore pressure of seabed through the pipeline and the dashed line and symbols

Fig. 6. Finite element mesh in the vicinity of the pipeline.

Fig. 7. Vertical distribution of normalized pore pressure p/p_0 versus z/h at different locations.

denote the results near the pipeline (the distance to the pipeline is 5 m). As illustrated in Fig. 7, the result of normalized pore pressure through the pipeline at any fluid node plotted in the figure is smaller than that near the pipeline; it may be because of undrained boundary conditions (see Section 2). As seabed depth increases, the normalized pore pressure decreases rapidly at the upper part of seabed, and then changes slightly at the lower part of the seabed. This conclusion agrees with the computation results of 3D pore-elastic model [24].

4.2. The distribution of pore pressure of seabed around submarine pipeline

Fig. 8 shows the distribution of normalized pore pressure p/p_0 of seabed in the vicinity of the buried pipeline with contour plots, values. The solid lines with values denote the results of existing pipeline and the dashed lines denote the ones of free seabed. As seen in Fig. 8, the existence of pipeline has a notable effect on seismic excess pore pressure of seabed around the pipeline. The further the distance from pipeline, the smaller the effect of pipeline. At a given distance away from the pipeline, the effect of existence of pipeline is slight and the distributions of both tend to be consistent.

Fig. 9 illustrates the distribution of normalized pore pressure around submarine pipeline. In Fig. 9, θ is the polar angle in the

Fig. 8. Distribution of normalized pore pressure p/p_0 of seabed in the vicinity of the buried pipeline.

Fig. 9. Vertical distribution of normalized pore pressure p/p_0 around submarine pipeline.

polar coordinate and it is measured positive in the counterclockwise direction of the pipeline. The value of normalized pore pressure at any fluid node is smaller than 1.0, but the maximum value of normalized pore pressure occurs near $\theta=60^\circ$ and the minimum value of that occurs near $\theta=300^\circ$. Fig. 9 clearly demonstrates the significant influence of elasto-plastic soil behavior on normalized pore pressure. It is observed that the results of normalized pore pressure are not axisymmetric along the pipeline, which may come from elasto-plastic soil behavior.

4.3. The comparison of normalized pore pressure with the present elasto-plastic model and elastic model

The effect of the present elasto-plastic soil model and elastic soil model on normalized pore pressure has been examined; the results are shown in Fig. 10. Fig. 10 clearly demonstrates the obvious differences of elasto-plastic soil behavior and elastic soil behavior on normalized pore pressure. It is noted that the maximum differences of normalized pore pressure between elastic and elasto-plastic soil model can be up to 50% of p_0 .

Fig. 11 shows the time history curves of excess pore pressure at the bottom and the top point of submarine pipeline with elasto-

Fig. 10. Vertical distribution of normalized pore pressure p/p_0 versus z/h with the present elasto-plastic soil model and elastic soil model.

Fig. 12. Vertical distribution of normalized pore pressure p/p_0 versus z/h with variable soil permeability.

Fig. 11. The time history curves of pore water pressure of seabed with elasto-plastic soil model.

plastic soil model. It shows the significant increase of excess pore pressure in the seabed. It should be noted that no excess pore pressure build-up can be simulated if linear elastic model was adopted in the analysis. It further demonstrates the significant influence of wave-induced pore pressure of seabed around submarine pipeline with elasto-plastic soil model.

4.4. Effects of soil permeability to the distribution of pore pressure along seabed depth

Soil permeability is one of the most important soil parameters in this respect, which is a measure of how fast the pore fluid may transfer through the soil matrix. Fig. 12 plots the vertical distributions of normalized pore pressure versus soil depth of various permeabilities ($k=1.0 \times 10^{-3}$ m/s, 1.0×10^{-4} m/s and 1.0×10^{-5} m/s) with elasto-plastic soil behavior. As shown in Fig. 12, the pore pressure with different soil permeability decreases as soil depth increases up to a certain depth, and then changes slightly nearly the lower part of seabed as soil depth increases.

4.5. Effects of shear modulus to the distribution of pore pressure along seabed depth

Soil shear modulus is another important soil parameter in the analysis of wave-induced seabed response. According to the mentioned description, the change of shear modulus of soil G_{eso}

Fig. 13. Vertical distribution of normalized pore pressure p/p_0 versus z/h with variable soil shear modulus.

leads to the change of G_{eso} . For convenience of description, G_{eso} is examined in this analysis. Fig. 13 shows the vertical distributions of normalized pore pressure versus soil depth of various G_{eso} ($G_{eso} = 1155$ kPa, 2310 kPa and 3465 kPa). As shown in Fig. 13, it can be observed that there is no big difference among different shear modulus of soil. They show the same trend with increase in the soil depth. The pore pressure decreases firstly as soil depth increases up to a certain depth, and then changes slightly until to the lower part of seabed.

4.6. Effects of wave period to the distribution of pore pressure along seabed depth

Wave period is an important variable in the determination of wavelength that has been involved in most wave characteristics. Fig. 14 illustrates the vertical distributions of normalized pore pressure versus soil depth for various values of wave period ($T=5.0$ s, 7.5 s and 10.0 s). Fig. 14 clearly demonstrates the significant influence of different wave periods on normalized pore pressure with elasto-plastic soil behavior. The pore pressure with different wave periods decreases as soil depth increases up to a certain depth (about $z/h=0.5$), and then changes slightly as soil depth increases. Furthermore, it is noted that as the wave periods increase, the normalized pore pressures have slight difference at the above part of seabed, but it increases normally in order at the under part of seabed as the wave periods increase.

Fig. 14. Vertical distribution of normalized pore pressure p/p_0 versus z/h with variable wave period.

Fig. 15. Vertical distribution of normalized pore pressure p/p_0 versus z/h with variable soil elasto-plastic parameter.

4.7. Effects of soil elasto-plastic parameter to the distribution of pore pressure along seabed depth

Soil stress–strain property is another characteristic that may affect the wave-induced pore pressure inside the seabed. As shown in Eq. (9), the volumetric strain change of soil affects the storage capacity in the pores, and then subsequently influences the pore pressure via the equation of mass conservation for the fluid phase. For the elasto-plastic model, the strain change can be divided into elastic and plastic part. As for the change of plastic strain, H_0 is one of the most important parameters.

Fig. 15 plots the distribution of normalized pore pressure versus soil depth for various values of $H_0 = 600, 700$ and 800 . As shown in the figure, the value of pore pressure at the fluid node plotted in the figure with various value of H_0 is almost the same; the difference among them can be neglected.

5. Conclusions

In this study, the three-dimensional poro-elasto-plastic Finite Element Method model with submarine pipelines was proposed. In this model, the Pastor Zeinkiewicz mark III elasto-plastic model is employed to describe soil plastic behavior of marine sediments under wave loading. Based on the numerical results presented above, the following conclusions can be drawn:

- (1) The 3D poro-elasto-plastic model proposed in describing the elasto-plastic properties of soil has noticeable difference with

elastic model. The poro-elasto-plastic behaviors of soil have more significant influence on wave-induced pore pressure of seabed around submarine pipeline.

- (2) With any variable soil characteristics, wave characteristics and elasto-plastic parameters examined in this analysis, the distribution of normalized pore pressures has the same trend: decreases rapidly at the upper part of seabed, and then changes slightly at the lower part of the seabed as the seabed depth increases.
- (3) The effect of permeability coefficient of soil on normalized pore pressure of seabed is remarkable. Small reduction of permeability coefficient leads to remarkable change of normalized pore pressure of seabed. Therefore, the pipelines may be protected using coarse material of high permeability to cover layers to dissipate the pore pressure.

Acknowledgments

The authors wish to express their gratitude to the supports from China National Natural Science Foundation for Young Scientists through the Grant 51308014, the supports from Specialized Research Fund for the Doctoral Program of Higher Education of China through the Grant 20121103120027 and Beijing City Board of Education Scientific Research Project on the project through the Grant km201410005021 are mostly grateful. The research is also supported by China Postdoctoral Science Foundation funded project (Project no. 2014M550581) and Beijing city Postdoctoral Science Foundation funded project (Project no. 201422-50).

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