# CHAPTER 170

An Analytical Model for Ocean Wave-Soil-Caisson Interaction Yau-Tang Tsai<sup>1</sup>, William G. McDougal<sup>2</sup>, and Charles K. Sollitt<sup>2</sup>

#### Abstract

An analytical model is developed to predict the soil responses induced by waves and caisson motion. The caisson is founded on a rubble bedding layer overlying a soil of finite depth. The responses are modeled by the Biot consolidation equations. Two approximations are employed to reduce mathematical difficulties: 1) a boundary layer approximation to decouple pore pressure and soil motion in the Biot equations and 2) a contact solution approximation for a thin elastic layer to address mixed-type boundary-value problems.

# 1. Introduction

Caisson-type structures are commonly employed as breakwaters, bulkheads, and seawalls. Critical failure areas for these types of structures are the toe and underlying foundation. These failures often result from wave-induced scouring, high porewater pressure, and large stresses developed in the foundation soil. These processes may lead to rapid and deep erosion at the toe as well as liquefaction of the foundation material. Hence, an understanding of the failure mechanisms for the foundation and an evaluation of the foundation stability are necessary.

Durand and Monkmeyer (1982), Liu (1985), and Dias and Monkmeyer (1986) have developed analytical solutions to estimate the waveinduced dynamic porewater pressure underneath a fixed caisson or a fixed plate founded on a porous rigid seabed. However, Stematiu and Stera (1985) pointed out that the structure-soil interaction is extremely significant in the foundation design of a caisson. This interaction results in high local stresses in the soil skeleton. Attempts to model the combined effect of porewater pressure and displacements were made by Lindenberg et al. (1982) and Stematiu and Stera (1985) using finite element methods. Finite element models have the disadvantage that they do not provide the physical insight into the problem which is often revealed in an analytical solution. The first steps toward an analytical solution were presented by Mynett and Mei (1982) for a caisson founded directly on the seabed, assuming the soil to be a poroelastic half-space. These assumptions limit the range of application of the model in some practical applications.

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Therefore, the objective of this paper is to develop an analytical model to estimate foundation stresses, porewater pressure, and structure motions for a wave-loaded caisson founded on a rubblemound bedding layer overlying a soil of finite depth.

#### 2. Formulation of the Model

The wave-soil-caisson interaction problem is depicted in Fig. 1, in which h is the still water depth, d is the thickness of the soil skeleton, b is the thickness of the rubble bedding layer, 2c' is the width of the caisson; and 2c is the width of the mound foundation on the mudline through which the displacement repsonse of the caisson completely transfers to the soil skeleton. 2c is approximated employing a 2V:1H slope (Bowles, 1982) and is termed the effective width of the caisson base.



## Figure 1. Definition sketch for the coordinate system and wave-soilcaisson system.

The development of the analytical model necessitates a number of assumptions. Among these are:

- 1. This is a two-dimensional problem.
- The water depth is constant and small amplitude wave theory is applicable.
- 3. The soil and wave responses are decoupled.
- 4. A rigid caisson is founded on a rubble bedding layer which is considered to be hydraulically permeable and mechanically stiff compared with the underlying soil.
- 5. There is no slip between the caisson and the rubble bedding layer. Caisson displacements are completely transferred to the seabed through the rubble bedding layer.

6. The soil stratum is assumed to be linearly poroelastic, isotropic, and is confined by a horizontal mudline above and a horizontal impermeable, rigid bed below.

## Governing Equation

Biot considation theory (Biot, 1941) is applicable to a shift saturated poroelastic medium. It has been successfully employed to model a variety of soil-wave interaction phenomena (Yamamoto et al., 1978; McDougal et al., 1981; and Mei and Foda, 1981). The Biot equations couple the motions of fluid and solid phases. This coupling increases the mathematical difficulty. Mei and Foda (1981) developed a boundary layer approximation which decouples the pore pressure and solid motion. Near the mudline there exists a boundary layer in which drainage is relatively unimpeded and there is significant relative motion between the fluid and the solid. Farther from the mudline there is little drainage so the fluid and solid tend to move in phase. This is termed the outer region problem, which can be reduced to solving a classical elastostatic problem. However, the boundary conditions on the mudline are not satisfied. Hence, corrections are made according to the boundary layer solutions so that the boundary conditions on the mudline are satisfied.

The governing equations for the outer region and boundary layer problems have been developed by Mei and Foda (1981). Their results are summarized in the following.

<u>Outer Region Problem</u> - The governing equations for the outer region problem were derived from the equations of momentum.  $()^{0}$  denotes a parameter in the outer region problem. For sandy sea beds the equations of momentum for the solid and fluid yields

$$\frac{\partial(\tau_{ij}^{\prime o} - p^{o}\delta_{ij})}{\partial x_{ij}} = 0$$
(1)

where  $\tau_{ij}^{o}$  is the effective stress,  $p^{o}$  is the porewater pressure,  $\delta_{ij}$  is the kronecker delta, and x, is the axis in the Cartesian coordinates system. Introducing the total stress  $\tau_{ij}^{o}$  to (1) yields

$$\frac{\partial \tau_{ij}^{o}}{\partial x_{i}} = 0 \tag{2}$$

For convenience, lengths and displacements are scaled by the effective caisson half-width, c, and stresses and pressures are scaled by the mudline pressure amplitude due to the free surface propagating waves,  $P_0$ . The dimensionless variables will be denoted by upper case letters.

Applying Hook's law to (2) yields the governing equations for the outer region problem.

$$\nabla^2 U^{\circ} + \frac{1}{1-2\nu} \frac{\partial}{\partial X} \left( \frac{\partial U^{\circ}}{\partial X} + \frac{\partial V^{\circ}}{\partial Y} \right) = 0$$
 (3a)

$$\nabla^{2} \nabla^{0} + \frac{1}{1 - 2\nu} \frac{\partial}{\partial Y} \left( \frac{\partial U^{0}}{\partial X} + \frac{\partial V^{0}}{\partial Y} \right) = 0$$
(3b)

in which  $U^O$  and  $V^O$  are the dimensionless horizontal and vertical displacements, respectively,  $\nu$  is Poisson's ratio, and X and Y are the dimensionless horizontal and vertical axes in the Cartesian coordinate system, respectively.

First the displacements are determined and then stresses in the soil skeleton may be obtained by applying Hook's law to the displacement solutions. The porewater pressure in the outer region is related to the total stresses by

$$P^{O} = -\frac{T_{xx}^{O} + T_{yy}^{O}}{2(1+m_{O})}$$
(4)

in which

$$m_{o} = \frac{\beta' n_{o}G}{1-2\nu}$$
(5)

and is a parameter indicating the relative stiffness between the solid and pore fluid. In (5)  $\beta$ ' is the combined air-water compressibility,  $n_{o}$  is the porosity, and G is the shear modulus.

<u>Boundary Layer Correction</u> - The governing equation for the boundary layer problem was derived from the equation of momentum and the storage equation. Again, ()<sup>b</sup> denotes a parameter in the boundary layer. The porewater pressure  $P^b$  in the boundary layer is governed by the one-dimensional Terzaghi consolidation equation

$$\frac{\partial P^{b}}{\partial T^{b}} = \frac{\partial^{2} P^{b}}{\partial (Y^{b})^{2}}$$
(6)

in which  $T^{\text{b}}$  is dimensionless time scaled by the wave frequency and  $Y^{\text{b}}$  is the vertical coordinate and scaled by the boundary layer thickness  $\delta$ .

The boundary layer correction is summarized as

$$\begin{pmatrix} \mathbf{p}^{\mathbf{b}} \\ \mathbf{T}_{\mathbf{x}\mathbf{x}}^{\mathbf{b}} \\ \mathbf{T}_{\mathbf{y}\mathbf{y}}^{\mathbf{b}} \\ \mathbf{T}_{\mathbf{y}\mathbf{y}}^{\mathbf{b}} \\ \mathbf{T}_{\mathbf{x}\mathbf{y}}^{\mathbf{b}} \end{pmatrix} = \begin{pmatrix} 1 \\ \frac{\nu}{1-\nu} \\ 1 \\ 0 \end{pmatrix} \mathbf{p}^{\mathbf{b}}$$
(7)

in which  $T_{xx}^{,b}$  and  $T_{yy}^{,b}$  are the effective dimensionless horizontal and vertical normal stresses, respectively, and  $T_{xy}^{,b}$  is the dimensionless shear stress. No shear stress correction is required in the boundary layer.

The ratio of the boundary layer thickness and the effective caisson half-width is denoted by

$$\varepsilon = \frac{\delta}{c}$$
 (8)

in which

$$\delta = \sqrt{\frac{\mathbf{k}^{*}}{\sigma_{\star}}} \left(\beta' \mathbf{n}_{0} + \frac{1}{G} \frac{1-2\nu}{2(1-\nu)}\right)^{1/2}$$
(9)

where k' is the intrinsic permeability and  $\sigma$  is the wave frequency. The displacement components for the boundary layer correction are at most of order  $\epsilon$  and can be neglected.

## Boundary Conditions

The boundary conditions for the wave-soil-caisson problem are:

- 1. Along the exposed portion of the mudline, the wave pressure and the total normal stress are continuous and the shear stress is negligible (McDougal et al., 1981).
- 2. Under the structure, the displacements are continuous at the caisson-soil interface.
- 3. Along the entire mudline, the wave-induced pore pressure is continuous.
- 4. The soil overlies a rigid bed at which no-slip and no normal fluid flow conditions are imposed.

These boundary conditions are summarized in Fig. 2.

Outer Region Problem - The soil displacement and the caisson motion are coupled. The wave forces along the exposed portion of the mudline induce soil stresses and displacements underneath the caisson. The wave forces on the caisson induce caisson motions which result in stresses and displacements in the soil away from the caisson. Linearity allows these two effects to be decoupled into scattering and radiation problems. In the scattering problem, the caisson is assumed to be fixed and the soil response is completely driven by the wave pressure on the mudline. The forces on the caisson determined in the scattering problem are applied on the caisson in the radiation problem. These forces result in caisson motion on an otherwise static seabed. The sum of these two components, the scattering problem and



Figure 2. Boundary conditions for the wave-soil-caisson system the radiation problem, yields the total response. This technique is illustrated in Fig. 3.



Figure 3. Decomposition of the wave-soil-caisson problem

<u>The Scattering Problem</u> - For the scattering problem the caisson is fixed and the mudline boundary conditions are

$$T_{xy}^{0}(X,0) = 0$$
 ;  $0 \le |X| \le \infty$  (10a)

$$T_{yy}^{0}(X,0) = -P_{w}(X)e^{-i\sigma t} \qquad ; 1 \le |X| \le \infty \qquad (10b)$$

 $U^{0}(X,0) = 0$  ;  $|X| \le 1$  (10c)

$$V^{O}(X,0) = 0$$
 ;  $|X| \le 1$  (10d)

This results in a mixed boundary-value problem. To solve this problem, a sequence of two solutions is sought: one which satisfies the stress boundary conditions (problem (a)) and then a second solution which satisfies the displacement boundary conditions (problem (b).

The boundary conditions for problem (a) and (b) are shown in Fig. 4. The displacements along the mudline,  ${}^{a}U^{O}(X,0)$  and  ${}^{a}V^{O}(X,0)$ , from the solution for the problem (a) are modified to provide the boundary conditions for problem (b). The first modification is that under the structure the displacements are set equal to zero. The second modification is based on the solution to the contact problem on a thin elastic layer developed by Alblas and Kuipers (1969). This solution provides appropriate displacements adjacent to the structure. Details are presented in Appendix I. The solution to problem (b) satisfies both stress and displacement conditions along the mudline. Thus, it is the solution for the scattering problem.

<u>The Radiation Problem</u> - The radiation problem is a moving caisson on an otherwise static soil. The response of the soil must satisfy the caission displacement conditions on the mudline. The caisson responds to both wave and soil loadings from the scattering problem and a restoring force in the radiation problem. The scattering loads are known but the motion is still unknown. A dynamic boundary condition must be prescribed to solve for the unknown caisson motion. This condition yields the amplitude of the caisson motion. Linearity allows the soil response to be decomposed into three problems corresponding to each of the three degrees of freedom of caisson motion. The caisson is assumed to be rigid and the effects of the structural damping in the caisson are assumed to be negligible compared with the soil.

Underneath the caisson the motions are specified while along the exposed portion of the mulline zero stress conditions apply. The boundary conditions for the radiation problem are depicted in Fig. 5. The respective boundary conditions along the mulline for the problems of surge, heave, and pitch may be defined by appropriate displacement and stress conditons. Displacement conditions over the entire upper boundary may be written by introducing  $U_f^{O}$ ,  $V_f^{O}$ , and  $V_f^{O}$ , which account for the unknown displacements along the exposed portion



problem (a)



# problem (b)

Figure 4. Boundary conditions for the scattering problem

of the mudline. Stress conditions for each degree of freedom of the caisson motion are also reasonably approximated as given by Alblas and Kuipers (1969 and 1970).

(1) surge

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$$T_{yy}^{0}(X,0) = 0 \qquad ; 0 \le |X| \le \infty$$
 (11a)

$$U^{o}(X,0) = U_{c} + {}^{1}U_{f}^{o}$$
;  $0 < |X| < \infty$  (11b)

(2) heave

 $T_{xy}^{O}(X,0) = 0$  ;  $0 < |X| < \infty$  (12a)

$$V^{o}(X,0) = V_{c} + {}^{2}V_{f}^{o}$$
;  $0 < |X| < \infty$  (12b)



Figure 5. Boundary conditions for the radiation problem

(3) pitch

$$T^{o}_{xy}(x,0) = 0 ; 0 \le |x| \le \infty (13a)$$
$$V^{o}(x,0) = -\alpha_{c}x + {}^{3}V^{o}_{f} ; 0 \le |x| \le \infty (13b)$$

These are no longer mixed-type boundary conditions, and the boundaryyalue problem can easily be solved. The displacements  $U_f^0$ ,  $V_f^0$ , and  $V_f^0$  can be evaluated by employing the thin soil layer contact problem solution technique presented in Appendix I.

$${}^{1} \mathbb{U}_{f}^{o} = \mathbb{U}_{c} \begin{cases} 1 - \tanh\left(2\sqrt{\frac{(1-\nu)(-1-X)}{\pi D}}\right) & ; -\infty < X < -1 \\ 1 - \tanh\left(2\sqrt{\frac{(1-\nu)(X-1)}{\pi D}}\right) & ; 1 < X < \infty \end{cases}$$
(14a)  
$${}^{2} \mathbb{V}_{f}^{o} = \mathbb{V}_{c} \begin{cases} 1 - \tanh\left(2\sqrt{\frac{A(-1-X)}{\pi D}}\right) & ; -\infty < X < -1 \\ 1 - \tanh\left(2\sqrt{\frac{A(X-1)}{\pi D}}\right) & ; 1 < X < \infty \end{cases}$$
(14b)

$${}^{3}V_{f}^{0} = \alpha_{c} \begin{cases} 1-\tanh\{2\sqrt{\frac{A(-1-X)}{\pi D}} (1 + i S'(0))\}; -\infty < X < -1 \\ \\ 1-\tanh\{2\sqrt{\frac{A(X-1)}{\pi D}} (1 + i S'(0)\}; 1 < X < \infty \end{cases}$$
(14c)

In (11b), (12b), and (13b),  $U_c$ ,  $V_c$ , and  $\alpha_c$  are the displacements of the caisson in surge, heave, and pitch, respectively. Positive  $\alpha_c$  is defined as counterclockwise. These displacements are unknown, but may be determined from the dynamic boundary conditions on the soil-caisson interface, i.e., the equation of motion for the caission.

$$\begin{pmatrix} \mathbf{m} & 0 & 0 \\ 0 & \mathbf{m} & 0 \\ 0 & 0 & \mathbf{I}_{\mathbf{m}} \end{pmatrix} \begin{pmatrix} \ddot{\mathbf{U}}_{\mathbf{c}} \\ \ddot{\mathbf{v}}_{\mathbf{c}} \\ \ddot{\mathbf{v}}_{\mathbf{c}} \end{pmatrix} = \begin{pmatrix} \mathbf{f}_{w1} \\ \mathbf{f}_{s1} \\ \mathbf{f}_{s2} \\ \mathbf{f}_{s2} \\ \mathbf{f}_{s3} \\ \mathbf{f}_{s3} \\ \mathbf{f}_{s3} \end{pmatrix}$$
(15)

where m and  $I_m$  are the mass and mass moment of inertia of the caisson, respectively,  $\ddot{v}_c$ ,  $\ddot{v}_c$ , and  $\ddot{\alpha}_c$  are the accelerations of surge, heave, and pitch of caisson motion,  $f_{wj}$  and  $f_{sj}$  are the wave and soil loads on the caisson determined in the scattering problem, and  $f_{rj}$  are the restoring forces on the caisson. The soil loads are obtained by integrating the stresses along the caisson-seabed interface.

The Boundary Layer Correction - Along the mudline, the summation of the pressure from the boundary correction and outer region solution is equal to the wave-induced porewater pressure. Along the rigid bed underlying the soil, the normal derivative of the pore pressure must be zero, i.e.,

at 
$$Y = 0$$
  $P^{0} + P^{b} = n_{0} P_{w}(X)$ ;  $0 \le |X| \le \infty$  (16a)  
at  $Y = D$   $\frac{\partial (P^{0} + P^{b})}{\partial Y} = 0$ ;  $0 \le |X| \le \infty$  (16b)

3. Solution to the Model

The poroelastic layer occupies the strip space  $-\infty < X < \infty$ ; 0 < Y < D. A Fourier transform is applied with respect to X in Eq. (3). It follows that the general solution for the outer region problem is in the form of Fourier transform

$$\hat{U}^{0}(s,Y) = a_{1}ch(sY) + a_{2}sh(sY) + a_{3}Ych(sY) + a_{4}Ysh(sY)$$
 (17a)

$$\hat{v}^{0}(s,Y) = -i\{a_{1}sh(sY) + a_{2}ch(sY) + a_{3}[Ysh(sY) - \frac{3-4\nu}{s}ch(sY)] + a_{4}[Ych(sY) - \frac{3-4\nu}{s}sh(sY)]\}$$
(17b)

in which s is a Fourier transform parameter and functions ch and sh are abbreviations of cosh and sinh. The coefficients  $a_1$ ,  $a_2$ ,  $a_3$ , and  $a_4$  may be quantified by applying boundary conditions for the scattering and radiation problems, respectively. The inverse Fourier transform of the solution can be evaluated with a fast Fourier transform.

For simple harmonic waves, the solution for the boundary layer problem can also be easily solved.

$$p^{b} = \left\{ \left[ n_{o} P_{w}(X) - P^{o} \right]_{Y=0} \right] \frac{\cosh\left[\frac{1-i}{\sqrt{2}} \frac{D-Y}{\epsilon}\right]}{\cosh\left(\frac{1-i}{\sqrt{2}} \frac{D}{\epsilon}\right)}$$

$$-\frac{1+i}{\sqrt{2}} \varepsilon \frac{\partial P^{0}}{\partial Y} \Big|_{Y=D} \frac{\sinh\left(\frac{1-i}{\sqrt{2}} \frac{Y}{\varepsilon}\right)}{\cosh\left(\frac{1-i}{\sqrt{2}} \frac{D}{\varepsilon}\right)} e^{-iT^{b}}$$
(18)

The superimposition of the solutions for the scattering problem, radiation problem, and boundary layer problem yields the total solution to the original problem. Since the response of the soil to waves and caisson motions is not readily apparent from the analytical solution, a numerical example will be presented to examine the solution behavior. For this examination the conditions given in Table 1 are assumed.

Contours of the soil response are presented in Fig. 6. These plots are for the wave crest at the face of the caisson (x = -c). Figures 6(a) and (b) show the horizontal and vertical displacements. The horizontal displacement is rather symmetrical with respect to the caisson center line. The surge motion of the caisson tends to dominate this displacement. The vertical displacement is antisymmetrical with respect to the caisson center line because the pitch motion is dominant in this displacement. However, the wave-induced vertical displacement is also significant near the caisson toe. Figures 6(c) through (e) show the shear stress, effective horizontal and vertical normal stresses, respectively. High stress concentrations occur at the caisson toe and heel. Both the shear stress and the effective horizontal normal stress decay with the depth much faster than the





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Wave Period Wave Height Water Depth Rubblemound Thickness c' m Im d	10 sec 6.1 m 12.2 m 1.5 m 6.4 m 64,240 kg 6,397,800 kg-m <sup>2</sup> 7.2 m (dimensionless depth = 1.0)
Rubblemound Thickness	1.5 m
. c'	6.4 m
m.	64,240 kg
I_	$6,397,800 \text{ kg-m}^2$
d	7.2 m (dimensionless depth = $1.0$ )
ν	0.15
G	$19.640 \text{ KN/m}^2$
n_	0.3
k <sup>0</sup>	$1.03 \times 10^{-8} \text{ m}^{4}/\text{sec}/\text{N}$
ß'	$4.53 \times 10^{-10} \text{ m}^2/\text{N}$
•	•

Table 1. Wave, caisson, and soil conditions.

effective vertical normal stress. Figure 6(f) shows the porewater pressure. Due to the influence of the impermeable rigid bed on the porewater pressure, higher porewater pressures are developed near the impermeable underlying bed rather than near the mudline.

#### 4. Conclusions

- 1. A linear, two-dimensional analytical model for wave-soil caisson interaction has been developed. The caisson is founded on a rubble bedding layer overlying linearly poroelastic soil of finite depth.
- 2. Soil displacements, stresses, and porewater pressure as well as caisson motion are predicted.
- 3. Two approximations are employed: a boundary layer approximation and a contact solution approximation for a thin elastic layer.
- 4. The pitch motion of the caisson develops the largest soil response while heave is the least significant.
- 5. The caisson motion induces much larger displacements, stresses, and porewater pressure in the soil than the wave alone. This indicates that a static analysis may significantly underestimate stresses.

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### Appendix I. Displacement Boundaries along the Exposed Mudline

A contact solution approximation has been developed for a rectangular block on a thin elastic layer by Alblas and Kuipers (1969). The boundary conditions along the upper boundary of the elastic material are of the mixed type. The displacement condition along the block elastic-layer interface is defined while the stress along the exposed portion of the upper surface are known. Under the assumption of a thin elastic layer, Alblas and Kuipers (1969) developed an approximate solution for the displacement of the elastic layer surface adjacent to the block.

heave

$$V'_{2}(X) = V_{0} \left[ 1 - 2 \sqrt{\frac{A(X-1)}{\pi D}} \right]$$
;  $0 < X-1 << 1$  (I.1)

pitch

$$V'_{3}(X) = \alpha_{c} \left\{ 1 - 2 \sqrt{\frac{A(X-1)}{\pi D}} \left[ 1 + i \hat{S}'_{-}(0) \right] \right\} ; 0 < X-1 << 1$$
 (I.2)

where  $A = 2(1-v)^2/(1-2v)$  and S'(0) is a function dependent on Poisson's ratio and the layer depth (Alblas and Kuipers, 1970). These solutions are only valid near the structure. Hence, a tanh function is introduced to modify (I.1) and (I.2) to yield approximate solutions which approach the original solutions near the structure and approach the undeformed upper surface at large distances. The tanh function also satisfies the no stress condition along the exposed portion of the elastic layer.

heave

$$\mathbb{V}_{2}(X) = \mathbb{V}_{c}\left\{1-\tanh\left[2\sqrt{\frac{A(X-1)}{\pi D}}\right]\right\} \qquad ; 1 \leq X < \infty \qquad (I.3)$$

pitch

$$V_{3}(X) = \alpha_{c} \left[ 1 - \tanh \left\{ 2 \sqrt{\frac{A(X-1)}{\pi D}} \left[ 1 + i \hat{S}'_{-}(0) \right] \right\} \right] ; 1 \le X \le \infty$$
 (I.4)

Alblas and Kuipers (1969) did not develop a solution for the surge of the block motion when the block is subjected to a horizontal load. Therefore, an approximate solutions for the horizontal displacement is developed from the solution for the heave, (I.1)

$$U'_{1}(X) = U_{c} \left[ 1 - 2 \sqrt{\frac{(1-\nu)(X-1)}{\pi D}} \right]$$
;  $0 \le X-1 \le 1$  (I.5)

Introducing the tanh function yields

$$U_1(X) = U_c \left[ 1 - t \operatorname{anh} \left( 2 \sqrt{\frac{(1-v)(X-1)}{\pi D}} \right) \right] ; 1 \le X \le \infty$$
 (I.6)

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