COMPARISON OF DIFFERENT OVERWASH MODELS IN SIMULATING TIDAL ANOMALIES IN LAKE CONJOLA, NSW, AUSTRALIA

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Abstract: *Lake Conjola in Southern, New South Wales, Australia experienced a significant flooding event around 9th April 2006 (Figure 2). We explain how the raised water level in the lake was caused by waves with large height and very long period pumping water into the lagoon across a berm separating the lagoon from the ocean.*

The simulation considering lake system as two nodes in series is carried out using different overwash discharge Q_{over}*- formulae viz, the wave pumping model (Nielsen, 2001; Callaghan, 2006; Thuy et al, 2011), the swash model of Baldock (2007, 2010) and the empirical overtopping models (Van der Meer & Janssen, 1995; Hedges & Reis, 1998; Pullen et al, 2007).*

The results show that the wave pump model provides the lowest overwash flow rate while the group of three empirical models similarly gives highest discharge. The simulated water levels using different overwash models show that the group of empirical models overestimates the water level. The wave pump model and the swash model are comparable and give good agreement with measured water levels. The wave pump model best performs with matching measured tidal range and lowest RMSE.

Key words: ***over wash discharge; wave pump model; swash model; over topping model***

1. INTRODUCTION

Lake Conjola in Southern, New South Wales, Australia experienced a significant flooding event around 9th April 2006 (Figure 2). At that time, very large waves with unusually long periods arriving from the south, seemed to be the main driving force for flooding through strong overwash flows. During this flooding, fresh water inflow and ocean surge were insignificant. This case is used to illustrate the importance of wave overwash as a driving force and to evaluate different existing formulae to determine Q_{over} for a natural Lake-sand barrier system.

2. LAKE CONJOLA MORPHOLOGY AND AVAILABLE DATA

Lake Conjola is located at 35°16’00’’S and 150°30’11’’E, about 200km South of Sydney (Figure 1). The lagoon surface area is ca 5.9km^2 (Allsop, 2009) and the catchment area is ca 145km^2. This system is classified as a predominantly open lake, being open 62% of time (GHD, 2012). The system consists of 2 lakes and a long channel. The main Lake Conjola with surface area ca 4.3km^2 is connected to the ocean by a shallow sandy channel around 3km long and about 1m deep. Berringer Lake, a smaller lake with surface area of ca 1.5 km^2, is located around 1.5km from the entrance. It is connected to the main channel via a relatively short and narrow passage (Haines & Vienot, 2007). The entrance to the lake located close by Cunjurong northern rock shelf, is rather shallow (1m depth) and narrow (30m wide). The entrance channel separates the large sand lobes and long sand spit (Figure 2).

Patterson Britton & Partners (1999) found that inlet closure events occurred eight times since 1937, and all closure events were related to washover during severe wave storms. The length of the over-washed berm is ca 300m (Figure 2), and the berm crest is 1.2-1.5m above mean sea level. The maximum velocity at the entrance in flooding season is ca 1 to 1.4m/s. The damping coefficient is estimated as D≈350 for normal conditions, this means that the friction term dominates the system.

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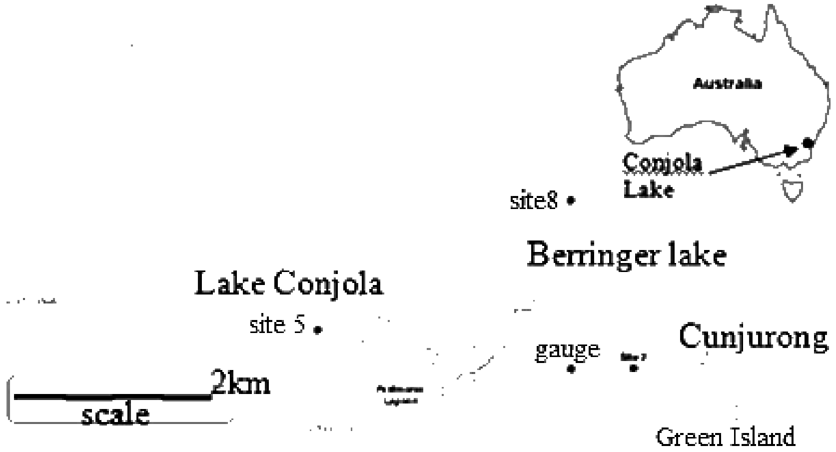

Figure 1: Location of Lake Conjola, NSW, Australia (sketch modified from source Allsop, 2009). For a close-up of the lower part of the system see Figures 2.9 and 2.10.

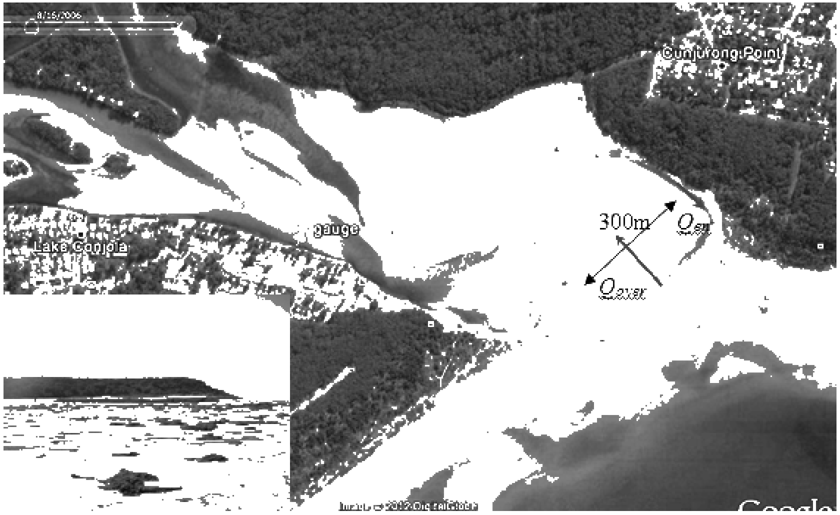

Figure 2: Closer view of Lake Conjola's entrance (16/8/2006) from Google Earth and location of tidal gauge. The Berm was overwashed in an extreme wave event on 9 April 2006 with debris left after event (bottom left), photo provided by Dave Hanslow.

Figure 3: Wave overwashed at the entrance of Lake Conjola (left), flooding in Caravan Park (yellow point in in Figure 2- right). Photos provided by Dave Hanslow.

During the large wave event in April 2006, the lake water level increased significantly overflowing its banks and flooding the roads and a caravan park (Figure 3). Photographs show that the berm was overtopped at some stage during 6th to 11th April 2006 and debris was left after event (Figure 2). The lagoon water levels were measured by a water level gauge located 1.3 km upstream from the entrance. Ocean tides were mixed, predominantly semi-diurnal with a range of ca 1m during the event. The peak tide measured in the lake at 5:30 am on 9 April 2006 was 1.31m, more than a meter above expected level, 0.5m higher than peak ocean tides (Figure 4). The corresponding anomaly was the highest ever recorded at Lake Conjola (Allsop, 2009; Kulmar & Hesse, 2008). A maximum offshore significant wave height, H_s, of *ca* 5 m occurred with the peak wave period (T_p) greater than 15 s. Wave direction during the event was from south to south east so that, the fronting berm was directly exposed to the waves, potentially resulting in significant wave setup and surf beat (Allsop, 2009; Kulmar & Hesse, 2008). A simultaneous oceanic surge measured at the ocean tide station (Eden) ranged between 0.3 m and 0.4 m.

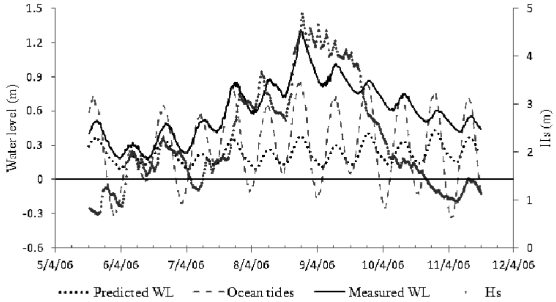

Figure 4. Available data for the event from 6th to 11th April 2006.

3. TWO NODE HYDRODYNAMIC MODEL

Traditionally, a one-node system was first investigated with the nominal surface area A_b being the whole bay surface area including Berringer Lake and Lake Conjola farther inside (Figure 1). However, results from one-node model by Thuy et al. (2011) show that the wave pump efficiency, ε, required to lift up whole lake level of ca 1m, is 9 times the suggested value by Nielsen et al. (2001) and Callaghan et al. (2006) for natural systems. This resulted in unrealistic velocity through the entrance and over the berm. Later they used a two nodes model to obtain a better agreement with measurements reflecting the real behaviour of the Lake Conjola system with ε = 0.035. In the two-node model, Lake 1 is the combination of the channel surface area and Lake Berringer while Lake 2 is the larger Lake Conjola farther inside (Figure 1). In addition, data from the December 2008 event (Figure 5) supports the two node approach. As can be seen, water levels measured at gauge were similar at site 8 in Berringer Lake while water levels at site 5 in Lake Conjola were well below and delayed. Therefore, the two-node model was chosen adopted.

In this case, $Q_f = 0$, due to insignificant rainfall. Hence the continuity equation for Lake 1 is

$$A_{b,1}\frac{\partial \eta_{b1}}{\partial t} = Q_{overwash} - Q_{en} + Q_{2-1} \quad (1)$$

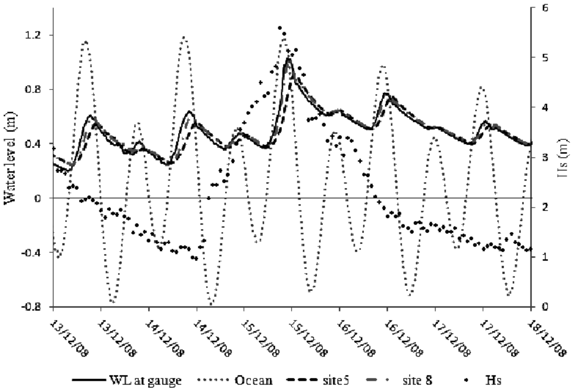

Figure 5. Water levels at different locations at Lake Conjola in the event Dec 2008.

$$Q_{2-1} = f\left(\eta_{b,2} - \eta_{b,1}\right) \quad (2)$$

and for Lake 2

$$A_{b,2}\frac{\partial \eta_{b,2}}{\partial t} = -Q_{2-1} \quad (3)$$

where $\eta_{b,1}$ and $\eta_{b,2}$ are the lake water surface elevations near the entrance and well within the lagoon and $Q_{2\text{-}1}$ is the discharge between the two nodes, positive from the inner node, see Figure 6. The overwash inflow Q_{over} across the berm is presented in 3.1 and the entrance outflow Q_{en} is obtained by the log-law friction model in 3.2. The solution method is a 4th order Runge-Kutta scheme.

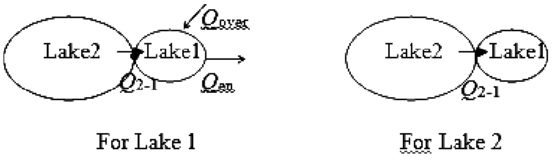

Figure 6. Sketch of calculation scheme for two node model.

3.1. The overwash flow over the Lake Conjola spit

Q_{over} is calculated and compared among 3 groups of models:

1) Wave pump model;

2) Overtopping related to swash truncation process (referred as swash model); and

3) Empirical overtopping models.

The suitable model will be chosen based on the best agreement with measurements.

1): The wave pump concept was introduced by Bruun & Viggosson (1977) for non-breaking waves and extended by Nielsen et al. (2001, 2008) for rip currents. This concept was applied successfully for studying atoll flushing by Callaghan et al. (2006). Thuy at al. (2011) applied the concept to simulate tidal anomalies for Lake Conjola for this event. However, the berm level of 2m used in calculation seemed too high (after observing another overwash event in July 2011). The overwash unit flow rate q_{over} [m^2/s] is estimated using the wave pump efficiency concept in Eq (4)

$$\varepsilon = \frac{\rho g \Delta q_{\text{over}}}{E_{\text{f}}} \quad (4)$$

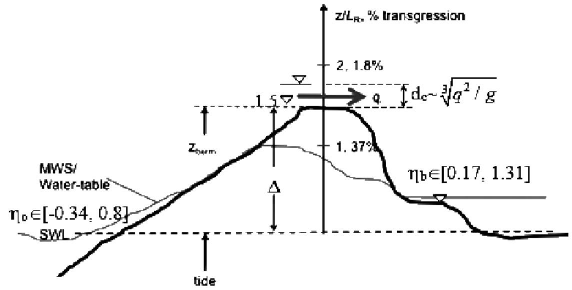

Figure 7: Notation and run-up scaling (L_R) for berm overwash. The % of waves transgressing corresponding to the ratio z/L_R in a Rayleigh distribution.

The wave pump efficiency is the ratio between the useful pump work [W/m] and the incoming wave energy flux E_f. Δ is the pumped height, ρ is the water density. The efficiency varies from around 3.5% in rip and atoll systems through to 40% – 50% on steep artificial ramps, cf. Figure 3 in Nielsen et al. (2008).

In a scenario where water is pumped over a berm crest, which is well above the offshore still water level, SWL, the lifting height Δ is simply taken as the berm crest height above the SWL as in Figure 7.

$$q_{\text{over}} = \varepsilon \frac{E_{\text{f}}}{\rho g \Delta} = \varepsilon \frac{E_{\text{f}}}{\rho g (z_{\text{berm}} - SWL)} \quad (5)$$

An extension to this is to include the critical flow depth, assuming that the flow across the berm is critical. z_{berm} is taken as 1.5m AHD from field measurement 2011 and 2012. The lifting height then becomes

$$\Delta = \left(z_{berm} - SWL\right) + \sqrt[3]{q_{\text{over}}^2 / g} \quad (6)$$

The wave energy flux is calculated using sine wave theory:

$$E_{\text{f}} = \frac{\rho g H_{0,\text{rms}}^2}{16} \frac{g T_{\text{p}}}{2\pi} \quad (7)$$

in which $H_{0,rms}$ is the off-shore root mean square wave height and T_p is the spectral peak wave period.

These data gathered by Nielsen et al. (2008) indicate a weak dependence of q_{over} on the relative freeboard, $\Delta / (\sqrt{H_0 L_0} \tan \beta_{\text{F}})$ (cf. Hanslow & Nielsen (1993) Figure 10).

2): Regarding breaking waves on a truncated beach, Baldock et al. (2005) found that the overtopping discharge derived by Shen & Meyer (1963), developed further by Peregrine & Williams (2001), underestimates Q_{over} by an order of magnitude compared to data and their numerical model based on Baldock & Holmes (1999). Later, Guard & Baldock (2007) developed a new swash model based on the non-linear shallow water equations and got realistic Q_{over} values by accounting for the shoreward momentum in surf-zone bores. Later, Baldock & Peiris (2010) expressed the relationship between dimensionless overtopping volume and dimensionless truncation point based on the Guard & Baldock (2007) solution with their k varying from 0 to 1.5. The parameter k represents different incoming mass and momentum fluxes at the seaward boundary (i.e., the point of bore collapse on a beach).

The overwash flow rate per unit length of berm q_{over} from Baldock & Peiris (2010) is:

$$q_{\text{over}} = \frac{R^2 V(E)}{2\sin(2\beta)T_p} \tag{8}$$

where β is the berm slope, R is run up limit height by Hunt (1956) given by

$$R = \begin{cases} \tan\beta\sqrt{H_{\text{o}}L_{\text{o}}} & \xi_{\text{o}} < 2 \\ H_{\text{o}}\sqrt{\dfrac{\pi}{\tan\beta}}\left(\dfrac{2\pi h_{\text{o}}}{L_{\text{o}}}\right)^{1/4} & \xi_{\text{o}} > 2 \end{cases} \tag{9}$$

with the surf similarity parameter given by $\xi_o = \dfrac{\tan\beta}{\sqrt{H_o / L_o}}$ and $V(E)$ is the total volume overtopping (including part of critical and supercritical flow) with $E = 2\Delta/R = 2(Z_{berm}\text{-SWL})$. $V(E)$ is calculated from the table derived by Hogg et al. (2011) with different k. k=0 corresponds to the analytical solution of Shen & Meyer (1963). k=1 corresponding to fully developed nearly uniform bores, i.e., a near horizontal water surface elevation behind the bore front, with both flow depth and flow velocity behind the front decreasing linearly with time. k>1 may represent the case, where continual incoming flow overruns the initial shoreline motion originating from bore collapse.

3): Laudier et al. (2011) applied several empirical overtopping models such as Van der Meer & Janssen (1995), Hedges & Reis (1998) and Pullen et al., (2007) for a barrier beach fronting a lagoon. They found that three of them worked similarly well with a reduction factor γ_r = 0.72-0.87 for two narrow- banded wave case; however, the European model (Pullen et al. 2007) had the best overall agreement with data and especially for broad-banded and double peak wave spectra. These empirical formulae are strongly dependent on slope geometry and used for breakwater with free overtopping. These overwash formulae are listed below:

Van der Meer & Janssen (1995), further referred as VJ model has an exponential form

$$\frac{q_{\text{over}}}{\sqrt{gH_{ST}^3}} = \frac{A}{\sqrt{\tan\beta}}\xi_o \exp\left[-B\frac{R_c}{\gamma_r \xi_o H_{ST}}\right] \qquad \xi_o \le 2$$
$$\frac{q_{\text{over}}}{\sqrt{gH_{ST}^3}} = C\xi_o \exp\left[-D\frac{R_c}{\gamma_r H_{ST}}\right] \qquad \xi_o > 2 \tag{10}$$

where H_{ST} is the significant wave height at the toe of the structure, which is hard to determine/define for natural sand barriers. Thus, as an alternative $R_{2\%}$ is utilized. T_{p} is the peak wave period; γ_r is the reduction factor for tuning, which depends on beach permeability, berm character, non-normal wave incidence and surface roughness. γ_r =0.72 - 0.87 is suggested by Laudier et al. (2011). The empirical coefficients A=0.06, B=5.2, C=0.2, D=2.6 are determined from laboratory data.

Pullen et al. (2007), referred further as the EU model, used the same type of formula but with different empirical coefficients; A=0.067, B=4.75, C=0.2, D=2.6. They used $T_{\text{m-1.0}}$ instead of T_{p}, between which the relationship is $T_{\text{p}} = 1.1\, T_{\text{m-1.0}}$.

Hedges & Reis (1998), referred to as the HR model is based on the physical equation of discharge over the weir,

$$\frac{q_{\text{over}}}{\sqrt{gR_{\max}^3}} = \begin{cases} A\left(1 - \dfrac{R_c}{\gamma_r R_{\max}}\right)^B & \text{for } 0 \le \dfrac{R_c}{\gamma_r R_{\max}} < 1 \\ 0 & \text{for } \dfrac{R_c}{\gamma_r R_{\max}} \ge 1 \end{cases} \tag{11}$$

where, they used $R_{\max 37\%}$ and gave the relationship between H_s and $R_{\max 37\%}$ depending on different ranges of ξ_o. The coefficients A and B are function of slope, cf. Laudier et al. (2011).

3.2 The flow rate through the Lake Conjola entrance channel

The entrance flow rate was modelled as a finite channel length model because the channel length L_c at Lake Conjola is significant of order 1300m. The model with friction given by the log-law in terms of the mean hydraulic depth, $R = A_c/P$ and the Nikuradse roughness k_{s} is used:

$$Q_{en} = \frac{A_c}{\kappa}\sqrt{g\frac{|\eta_o - \eta_b|}{L_c}R}\ln\frac{12R}{k_s}\,\mathrm{sign}(\eta_o - \eta_b) \quad (12)$$

4. APPLICATION, RESULTS AND DISCUSSION

4.1 Overwash flow rate at Lake Conjola

The overwash flow rate per unit width q_{over} by different methods is compared in Figure 8 based on ocean tides, z_{berm} and wave information. The wave pump efficiency ε=0.035 is chosen for Lake Conjola as suggested by Nielsen et al. (2001, 2008), Callaghan et al. (2006) and Thuy et al. (2011). For the swash model, k=1 is chosen as it provided a good description for usual conditions and corresponding to fully developed nearly uniform bores as suggested by Guard & Baldock (2007). An average reduction factor γ_r = 0.78 is applied for three empirical overtopping models as suggested by Laudier et al. (2011).

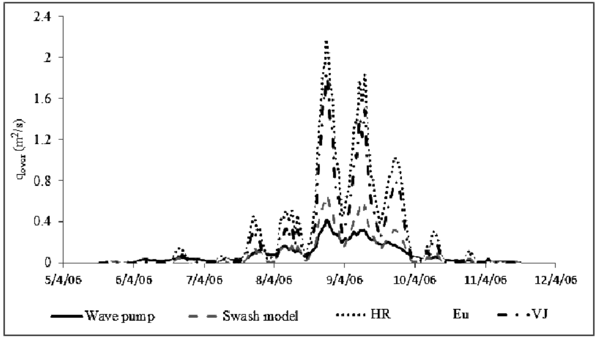

Figure 8: Results of overwash flow rate q_{over}(m^2/s) by different models: wave pump model with ε = 0.035, swash model with k = 1, empirical HR, Eu and VJ model with reduction factor γ_r = 0.78.

As can be seen, the three empirical models similarly give higher overtopping flow rates, by a factor of 5 compared to the lowest q_{over} from wave pump model, and a factor of 3 compared to the swash model. All models are active when H_s>2.5m and q_{over} is almost zero for H_s<2.5m. The q_{over} predicting was utilized to force the two node model to calculated bay water levels. The constant entrance cross section is chosen for simulation period with a rectangular shape of: 30m width and 1m depth. These dimensions are based on measurements from Google Earth images, field surveys in 1993, 2008, 2011 and 2012 as well as other related reports.

4.2 Comparison of water levels from different Q$_{over}$ models

Simulated water levels using different Q_{over} models are presented in Figure 8. In general a group of empirical models overestimate the water level especially during the period of intensive wave activity. Wave pump model and swash model are close to measurements, but similarly underestimate water levels. Wave pump model has better agreement with measurements compared to other models.

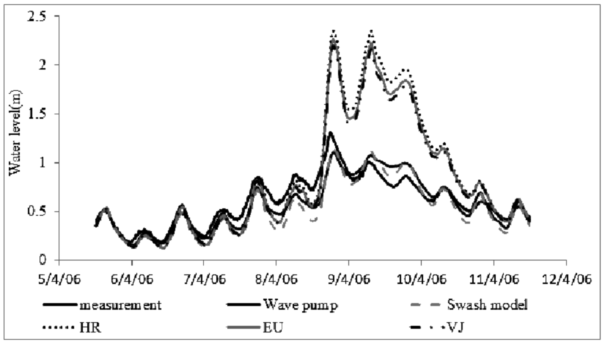

Figure 9: Results of water levels from different overtopping models: wave pump model with ε = 0.035, swash model with k = 1, empirical HR, Eu and VJ model with reduction factor γ_r = 0.78.

In particular, the three empirical models similarly predict 1 to 1.5m above measured water levels from late 8/4 to 10/4/2006 corresponding to the largest values of Q_{over} (Figure 8). The HR model persistently gives higher high water levels than the other two models in this group. To match the peak levels these models require a much larger cross section area of the order 80m width with the same 1m depth. Such large throat cross section area has not been observed before. In addition, the tidal range is still very large, a factor 2 to 3 of the measured tidal range even if the peak is matched. Therefore these empirical models are not suitable for this case. To improve the results, the reduction factor γ_r needs to be decreased together with adjustments of the channel cross section.

The wave pump model and swash model are similar in predicting same high water levels and same peak level even though wave pump model provide less overwash discharge than swash model (Figure). Both model underestimate ca 0.2m of water levels during 8/4/2006 compared to measurements. However, the wave pump model matches measured tidal range and has a better agreement with measurements at low tides. Therefore, overall the wave pump model (ε=0.035) has better fit with measurements compared to other models with RMSE of 0.1m and swash model (k=1) is the second one with RMSE of 0.15m. To get further better agreement with measurement both models need to have slightly higher values of ε and k coefficient, as well as varying cross section in time.

Conclusion

Case of inland flooding at Lake Conjola illustrates of the importance of wave overwash as a driving force. The two nodes model provides a good agreement with measurements, and reflects the real behaviour for the Lake Conjola system.

Three types of overwash models were investigated with calibration parameters based on previous studies. ε=0.035 is chosen for wave pump model, k=1 for swash model and γr = 0.78 is applied for three empirical overtopping models.

The empirical overtopping models greatly overestimate the water level corresponding to highest overwash discharge. The wave pump model and the swash model are comparable and give good agreement with measured water levels. However, the wave pump model is suggested for similar cases because the model performs best matching measured tidal range and lowest RMSE.

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