OCEAN DRIVEN FLOODING OF A COASTAL LAKE

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Analysis of Lake Conjola flooding in April 2006, provided in this paper, attributes it to waves pumping water over a 300 m long beach berm and into Lake Conjola. This overwash, generated by the medium wave height swell occurring during this flooding, was able to lift the lake levels near the entrance, persistently over several tidal cycles, to well above the ocean water levels . The wave pump model was used to model this flooding. Lake Conjola water storage and dynamics were modelled by using a two-node continuity based model that a change in storage in time is driven by the net inflow to a node and these nodes and the ocean are linked by log-law. The extents of these two nodes were established from previous water surface measurements. While the qualitative flood behavior was reproduced by this remarkably simple model, the peak flood level was not satisfactorily predicted when using literature values for model turning parameters. One reason for this mismatch was that the waves pumped against a head including critical flow on the beach berm. Based on recent images of Lake Conjola wave overwash events, it may be concluded that pumping against critical flow is too harsh. Removing this from the model has halved the gap between the measurements and predictions. However, more research is definitely required to establish what components should be included in the hydraulic head pumped against.

Keywords: coastal flooding, wave pumping, overwash, continuity modelling

INTRODUCTION

This paper examines the April 2006 flooding event at Lake Conjola, located on the south coast of New South Wales, Australia (see figs 1 and 3). This flooding had caused an inundation of the foreshore caravan park, at which the peak water level was about 1.31 m above mean sea level (MSL), and well above its typical elevation of between zero and 0.4 m MSL. This flooding occurred during sunny conditions and more particular, without any catchment rainfall several days before and during this flooding. This rules out rainfall and runoff being responsible for this flooding event. Further, there are no regulation structures (e.g., dams, weirs) within Lake Conjola catchment and hence, flooding from the controlled discharges is not possible. The flooding was observed during a period of energetic ocean swell arriving from the southern ocean, indicating an offshore wave direction near perpendicular to the shoreline/berm separating Lake Conjola from the Tasman Sea, see fig 1. This suggests a wave-driven type flooding mechanism

Lake Conjola temporal level variation was observed by local residents as rising with increasing wave action, albeit 'still tidal' (more precisely, modulated at tidal frequencies), indicating a link between the ocean water level and this flooding. However, as Lake Conjola levels were, during part of this flooding, above measured ocean levels for several days (ruling out flooding from resonating long waves from the incident short waves) and the lake was discharging towards the ocean, the dominant driving mechanism was not ocean/lake surface gradients or atmospheric pressure (including meto-tsunami) or wind setup or wave setup (Callaghan et al., 2013; Dunn, 2001; Dunn et al., 2000; Moura et al., 2013; Nielsen, 2009) or all combined.

Searching for the primary mechanism generating Lake Conjola flooding in April 2006 is the focus of this paper. In particular, we examine wave-based causes for flooding. It is hypothesized that swell waves with medium heights and long periods (fig 2), generated from a low pressure system south of New Zealand, pumped water across the beach berm fronting Lake Conjola. The remainder of this paper is divided into three parts. The first part develops a two-node continuity based model to model the lake water surfaces and discharges and then compares its predictions to the measurements. The second part will discuss possible model improvements, and the final part provides a brief summary of this work.

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Figure 1—Left: the location of Lake Conjola on the coast of New South Wales, Australia. Right: the entrance channel and beach berm of Lake Conjola in August 2006, images from Google Earth.

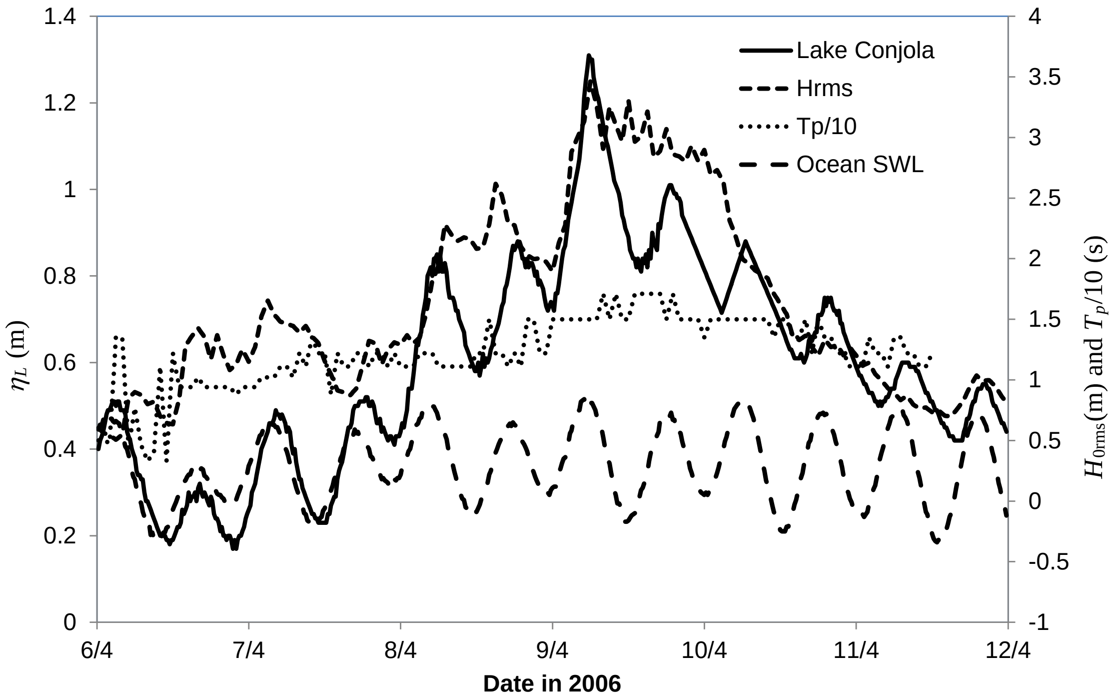

Figure 2—Lake Conjola water surface level near the caravan park (η_1, continuous line, left axis), the offshore root mean square wave height (H_{0rms}, short dashed line, right axis), peak wave period (T_p, dotted line divided by 10, right axis) and ocean surface level (long dashed, left axis) for April 2006 flooding.

LAKE DYNAMIC MODEL INCLUDING A WAVE PUMP

A transparent lake/ocean model was developed to elucidate the April 2006 flooding event. This model consists of continuity nodes using

$$\mathsf{A}\frac{\mathrm{d}\eta}{\mathrm{d}t} = \Sigma Q \tag{1}$$

where A is the surface area associated with a particular node with elevation η (positive upwards) and net inflow ΣQ. The lake was schematised using two nodes (fig 3) based on previous field measurements indicating that the western region fluctuates approximately uniformly with less range compared to the remainder of Lake Conjola (Vu, 2014). These data were also used to estimate the equivalent sand grain roughness to describe the velocity profile in the entrance channel, see equation (4).

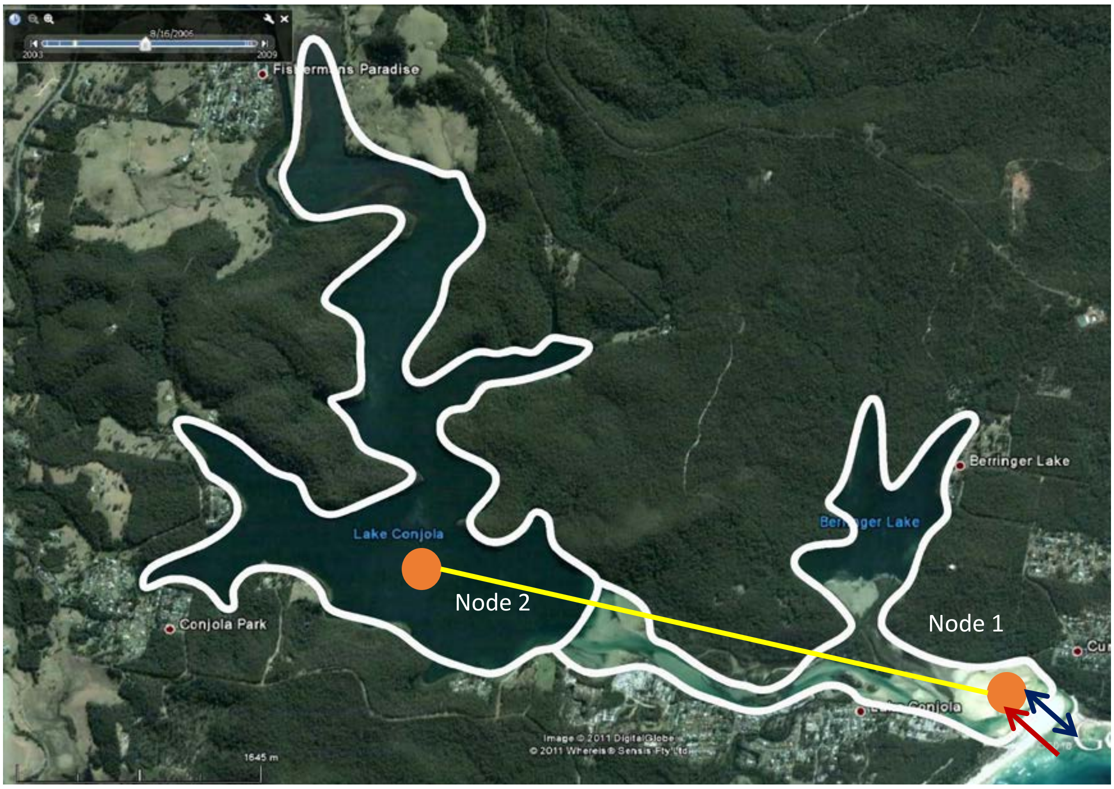

Figure 3—Two-node continuity model for Lake Conjola, including allowance for channel exchange between lake nodes and the ocean and inflow from waves pumping water over the beach berm. Image from Google Earth.

Node 1 was taken as the eastern part of Lake Conjola (fig 3, the channel surface area and Berringer lake, ca 1.5 km from the entrance) and has inflow from node 2 (Q_{2-1}), waves pumping water over the beach berm (Q_{overwash}) and outflow to the ocean (Q_{entrance}). Combing this with equation (1) gives

$$\mathsf{A}_1 \frac{\mathrm{d}\eta_1}{\mathrm{d}t} = Q_{\text{overwash}} - Q_{\text{entrance}} + Q_{2-1} \tag{2}$$

where $Q_{2-1} = f(\eta_2 - \eta_1)$. The western part of Lake Conjola or node 2 then is

$$\mathsf{A}_2 \frac{\mathrm{d}\eta_2}{\mathrm{d}t} = -Q_{2-1}\,. \tag{3}$$

The discharge between these nodes and the ocean ($Q_{2\text{-}1}$ and Q_{entrance}) was estimated using a friction slope relationship based on this version of the log-law (Callaghan, 2012; Callaghan, 2014)

$$Q_{\text{a to b}} = \mathsf{A}_{\text{ab}} \frac{\sqrt{g \frac{|\eta_{\text{a}} - \eta_{\text{b}}|}{L_{\text{ab}}} \frac{\mathsf{A}_{\text{ab}}}{\mathsf{P}_{\text{ab}}}}}{\kappa} \ln \frac{12 \frac{\mathsf{A}_{\text{ab}}}{\mathsf{P}_{\text{ab}}}}{k_s} \operatorname{sign}(\eta_{\text{a}} - \eta_{\text{b}}) \tag{4}$$

where A_{ab}, P_{ab}, L_{ab} and k_s are the channel flow area, wetted perimeter, length and equivalent sand roughness respectively, $\kappa \approx 0.4$ is the von Kármán's constant and g is the gravitational acceleration.

The overwash discharge was estimated using the wave pump concept. Bruun and Viggoson (1977) first introduced the non-breaking wave pump model that was extended to rip cells by Nielsen et al. (2001). Callaghan et al. (2006) applied Nielsen et al. (2001) model to atoll flushing. Nielsen et al. (2008) provided comprehensive measurements for wave pump efficiencies ε (fig 4). That is, the single tuning parameter in the wave pump model has been constrained by Nielsen et al. (2008)'s measurements. Wave pump efficiencies on steep ramps of 40° to 45° were found to be up to a half where as they are less than 0.1 for flat ramps less than 30° and ca 0.035 for two rip currents and one atoll lagoon flushing measurements.

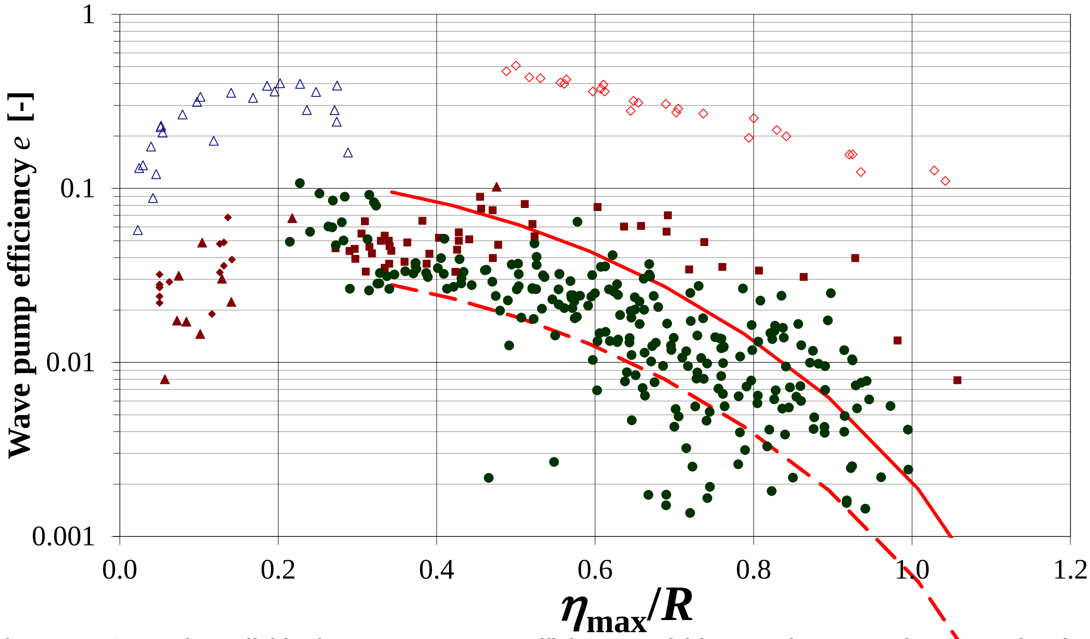

Figure 4— Currently available data on wave pump efficiency on fairly smooth ramps. Blue open triangles: Gourlay (1996) 45° slope, red open diamond: Kofoed et al. (2006) 40° slope, dark red solid square Wilson (1997) 1:5 slope; dark green solid circle Baldock et al. (2005) 1:10 slope, dark red solid diamond: field data from rip currents, dark red solid triangle Bruun and Viggoson (1977). Results from the swash model of Guard and Baldock (2007), 1:10 slope: red solid line $H_0/L_0 = 0.006$; red dashed line $H_0/L_0 = 0.07$. Data and model results from Nielsen et al. (2008).

In analytical terms, the wave pump discharge is

$$Q_{\text{overwash}} = \varepsilon \times \frac{E_f \times \int_0^L \cos\left[\theta_{berm}(s) - \theta_0\right] ds}{\rho g \Delta}, \tag{5}$$

where L is the berm length over which overwash occurs, E_f is the offshore wave energy flux and assumed to be constant along the berm, Δ is the pumped height, s is the curvilinear axis running along the shoreline/berm, $\theta_{berm}(s)$ is the bearing of the shoreline/berm normal, θ_0 is the deepwater wave direction and ρ is the seawater density. The wave energy flux was estimated using sine waves (Nielsen, 2009) based on the offshore (deepwater) root mean square wave height, $H_{0\text{rms}}$ and peak wave period T_p, i.e.,

$$E_f = \frac{1}{8}\rho g H_{0\text{rms}}^2 \times \frac{1}{2}\frac{gT_p}{2\pi}. \tag{6}$$

The pumped height was taken from the ocean still water level (η_0) up to the berm crest elevation (z_{berm}) plus the specific energy head (E_c) at critical flow depth (d_c) for Q_{overwash} (Elger et al., 2012) using the argument that the water depth is non-zero as waves are pumping water into the lake and that critical flow is a simple steady flow concept that is dependent exclusively on Q_{overwash}, i.e.,

$$E_c = 1.5 \times d_c = 1.5 \times \sqrt[3]{\frac{Q_{\text{overwash}}^2}{L^2 g}}, \tag{7}$$

and hence, the pumping height is

$$\Delta = z_{\text{berm}} + 1.5 \times \sqrt[3]{\frac{Q_{\text{overwash}}^2}{L^2 g}} - \eta_o. \tag{8}$$

Combining these equations yields

$$Q_{\text{overwash}} = \frac{\varepsilon g H_{0rms}^2 T_p \int_0^L \cos\left[\theta_{berm}(s) - \theta_0\right] \mathrm{d}s}{32\pi \left(z_{\text{berm}} + 1.5 \times \sqrt[3]{\frac{Q_{\text{overwash}}^2}{L^2 g}} - \eta_o \right)}, \tag{9}$$

which is implicit with respect to Q_{overwash}. However, equation (9) can be quickly solved by iterating in the current form with an initial value of $Q_{\text{overwash}} = 0$. The berm geometry (i.e., crest elevation and bearing) was held constant during the April 2006 storm. Wave directions during this storm are approximately shore normal and hence

$$\int_0^L \cos\left[\theta_{berm}(s) - \theta_0\right] \mathrm{d}s \approx L. \tag{10}$$

In this particular application, the berm crest level is well above the maximum lake flood level and consequently the need to introduced critical flow depth across the berm. This means that Q_{overwash} should be known previously known a priori. Thus, the system being solved consists of two nonlinear ordinary differential equations (nonlinearity introduced through using the log-law). To improved the numerical stability and accuracy, these equations are solved by the 4th order Runge-Kutta scheme (Press et al., 1992). The model predictions for η_1 (fig 5, short dashed line) indicates the wave pump was able to lift the lake levels above those measured in the ocean during this event. We have included predictions from a model forced only by ocean tides has also been included in this paper. These two simulations were started with lake levels of 0.2 m and 2 m MSL. In reference to these predictions, the first point to note is that the initial lake level is inconsequential by the 6th of April and hence, does not impact on predictions during this flooding event (in both gravity only and gravity plus wave overwash models). The second point is that spring/neap tidal pumping is unable to lift the lake level above that of the ocean as we expected but included in this paper. The third point is that during the storm period when there was no overwash (e.g., 6th April), all models matched well with the measured lake levels, confirming right orders of channel and roughness parameters determined. Finally, the wave overwash required to lift lake levels above that of the ocean was also concluded by Callaghan et al. (2006) for a continuous rim atoll lagoon.

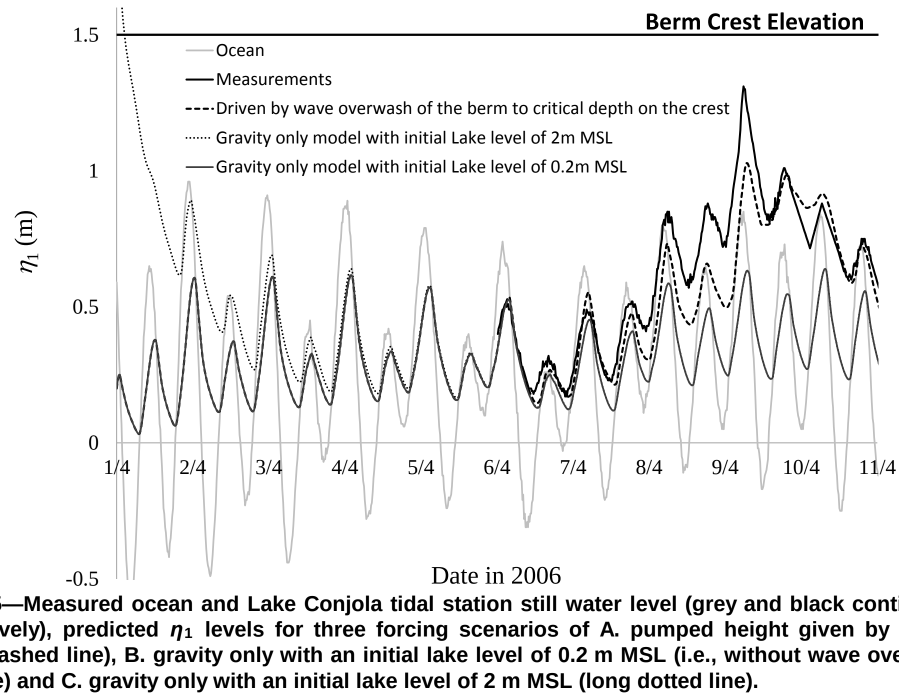

Figure 5—Measured ocean and Lake Conjola tidal station still water level (grey and black continuous lines respectively), predicted η_1 levels for three forcing scenarios of A. pumped height given by equation (8) (short dashed line), B. gravity only with an initial lake level of 0.2 m MSL (i.e., without wave overwash, dark grey line) and C. gravity only with an initial lake level of 2 m MSL (long dotted line).

IMPROVING PREDICTIONS

The Lake Conjola predictions for η_1, where wave pumping is used to lift the lake above the ocean still water level, indicate that waves are potentially involved in this flooding by berm overwash. Nevertheless, there are two drawbacks of this model when they are compared to measurements. The first short-coming is that if the wave pump efficiency is used to calibrate the model to match the maximum peak flood level (fig 6), an under-prediction before and over-prediction after the maximum peak flood level occurs. One possible reason for this mismatch could be that the ocean entrance channel increased in size during this storm (Vu, 2014). The second short-coming is timing of model peaks, which are generally matching well except for the largest peak that has a phase shift with that of the model prediction. This probably has to do with modelling Lake Conjola using two nodes only, each assuming constant water surface level. These two short-comings are fundamental in nature and require significantly more complex modelling approaches if they are to be addressed. A testing has done but not shown here for a three-node model marginally improved predictions. However, it is still possible to investigate the quantitative differences between measurements and predictions around the maximum flood peak. From Nielsen et al. (2008) (fig 4) and wave parameters, pumping efficiencies should range between 0.02 through to 0.05. Consequently, using 0.059 to calibrate this model to match the maximum flood peak is inconsistent with previous wave pump efficiency measurements. Hence, we look for other mechanisms to fully explain this flooding event.

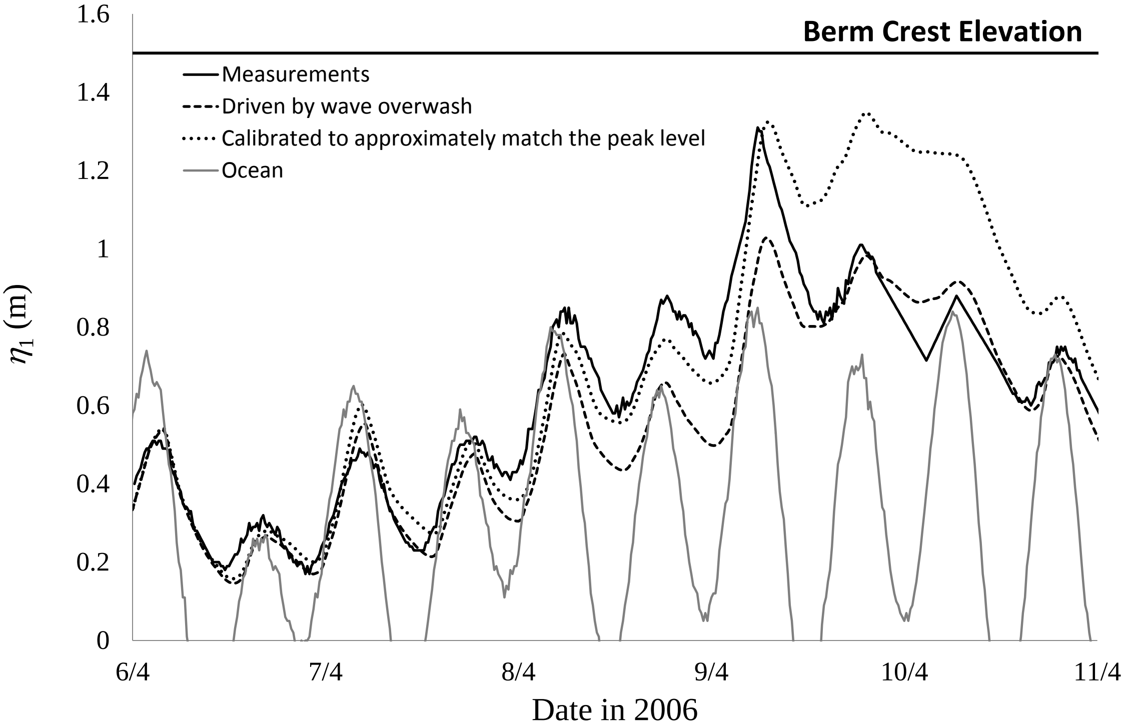

Figure 6—Measured ocean and Lake Conjola tidal station still water level (grey and black continuous lines respectively), predicted η_1 levels for one forcing scenarios of pumped height given by equation (8) using pumping efficiency of ε = 0.035 (dashed line) and ε = 0.059 (dotted line).

It is often argued that wave setup occurs in ocean entrances. This has been disproved for a large sized trained entrance (Gold Coast Seaway, Nielsen, 2009), a medium sized trained entrance (Brunswick River in Australia, Dunn, 2001; Dunn et al., 2000), a small sized trained entrance (Mooball Creek in Australia,Callaghan et al., 2013; Moura et al., 2013) and a small sized untrained entrance (Cudgera Creek in Australia, Callaghan et al., 2013; Moura et al., 2013). Nevertheless, we have included a scenario for completion, where this continuity based model is forced by gravity and wave setup in the entrance. Wave setup was modelled after Nielsen (2009) using

$$\delta_{\eta,\mathrm{setup}}(t) = \frac{0.4H_{\mathrm{orms}}(t)}{1+10\dfrac{h(t)}{H_{\mathrm{orms}}(t)}}, \tag{11}$$

where $h = (\eta_1 + \eta_o)/2 - z_{\text{chnl}}$, with predictions (fig 7, long dashed line) failing to lift the lake surface water level above that observed in the ocean.

The predictions of the model may be improved by adjusting the channel roughness parameter between nodes 1 and 2. However, the equivalent sand roughness used in this channel was obtained from previous measurements and hence are constrained by that calibration.

During May 2014, another event, although smaller than April 2006, occurred where waves overtopped the beach berm. The images of this event were obtained by local resident Brendon Wood (fig 8). These photos do not support the steady flow assumption of critical flow across the beach berm. While the mean water depth is not zero, one way of testing how important getting this depth correct in the wave pump model is by pumping against

$$\Delta = z_{\text{berm}} - \eta_o \,. \tag{12}$$

Reducing the pumping height will increase the wave overwash and hence, lift further the lake water level and in this case, it halved the gap between measured and modelled (fig 7, dotted line).

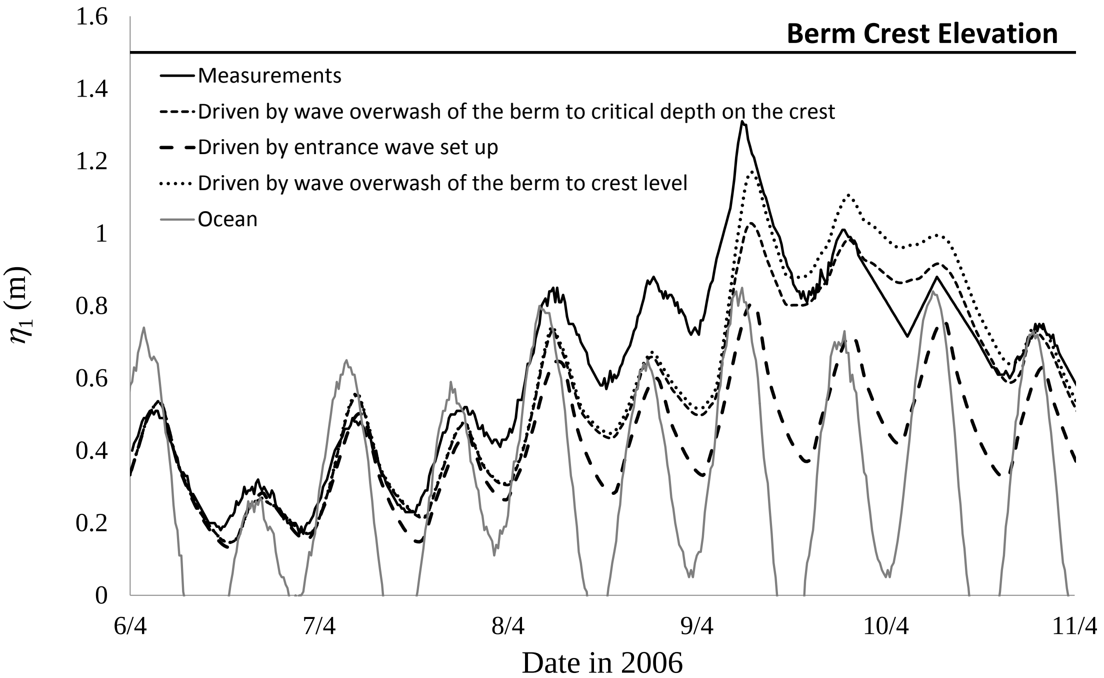

Figure 7—Measured ocean and Lake Conjola tidal station still water level (grey and black continuous lines respectively), predicted η_1 levels for three forcing scenarios of A. pumped height given by equation (8) (short dashed line), B. pumped height being $\Delta = z_{\text{berm}} - \eta_o$ (dotted line) and C. pump model replaced with an allowance of wave setup in the entrance (long dashed line).

Figure 8—Images of wave overwash at Lake Conjola between May 5 and 7, 2014. Images taken by Brendon Wood.

SUMMARY

Property flooding from Lake Conjola in April 2006 (figs 1 and 3) occurred in the absence of rainfall and the lake peak surface water level was well above that measured in the nearby ocean. The lake oscillations excluded the time periods associated with lake seiches (ruling out wind driven lake setup) and were persistently above ocean levels for more than three days, ruling out flooding from resonating long waves from the incident short waves or meteotsunami. During this period where Lake Conjola water levels were above the ocean, 15 s southerly swell occurred with root mean square wave heights between 1.5 m to 3.75 m, with the maximum peak in flooding occurring near the peak wave height (fig 2). Nevertheless, the lake levels were tidally modulated.

Based on the field measurements and the modelling results, it may be concluded that the flooding at Lake Conjola during early April 2006 was principally forced by waves pumping ocean water over the beach berm (fig 8) into Lake Conjola. Although the two node model is very simple in its form, it is able to simulate semi-realistic flooding behaviour in Lake Conjola. This was expected as similar approaches at Manihiki atoll lagoon using a correspondingly simplistic approach also adequately reproduced measurements (Callaghan et al., 2006; Sugandika et al., 2013). Model improvements were discussed in general terms (i.e., what more complex models would need to account for) and in terms of this particular model (refining the pumping height).

The flooding at Lake Conjola highlights the need for coastal engineers, managers and planners, which are often based on rainfall and ocean surge contributions, that close attention should be to paid to overwash when low and long beach berms are present.

REFERENCES

Baldock, T.E., Hughes, M.G., Day, K. and Louys, J., 2005. Swash overtopping and sediment overwash on a truncated beach. *Coastal Engineering*, 52(7): 633-645.

Bruun, P., F. and Viggoson, G., 1977. The wave pump: Conversion of wave energy to current energy. *Journal of the Waterway, Port, Coastal and Ocean Division*, 103(WW4): 449-469.

Callaghan, D.P., CIVL3141/2012, Catchment hydraulics: Open channel flow and design, Lecture Slides, 260 pp.

Callaghan, D.P., CIVL2131/2014, Fluid mechanics for civil and environmental engineers, Lecture Slides, 253 pp.

Callaghan, D.P., Nielsen, P., Baldock, T.E., Moura, T., Olfateh, M. and Golshani, A., 2013. *Wave setup and tides at the trained and untrained river entrances of hastings point and pottsville*, The University of Queensland, St Lucia, QLD 4072.

Callaghan, D.P., Nielsen, P., Cartwright, N., Gourlay, M.R. and Baldock, T.E., 2006. Atoll lagoon flushing forced by waves. *Coastal Engineering*, 53(8): 691-704.

Dunn, S.L., 2001. *Wave setup in river entrances*. Ph.D. Thesis, The University of Queensland, St. Lucia, Qld., 175 pp.

Dunn, S.L., Nielsen, P., Madsen, P.A. and Evans, P., 2000. Wave setup in river entrances. *Proceedings of the 27th International Conference on Coastal Engineering*, Sydney, Australia, 3432-3445.

Elger, D.F., Williams, B.C., Crowe, C.T. and Roberson, J.A., 2012. *Engineering fluid mechanics*. Wiley, Hoboken, USA, 673 pp.

Gourlay, M.R., 1996. Wave set-up on coral reefs. 1. Set-up and wave generated flows on an idealised two dimensional horizontal reef. *Coastal Engineering*, 27(3-4): 161-193.

Guard, P.A. and Baldock, T.E., 2007. The influence of seaward boundary conditions on swash zone hydrodynamics. *Coastal Engineering*, 54(4): 321-331.

Kofoed, J.P., Frigaard, P., Friis-Madsen, E. and Sørensen, H.C., 2006. Prototype testing of the wave energy converter wave dragon. *Renewable Energy*, 31(2): 181-189.

Moura, T., Olfateh, M., Callaghan, D.P., Nielsen, P., You, Y.G. and Baldock, T.E., 2013. *Tidal amplitude and wave setup in trained and untrained river entrances*, Coasts and Ports, Sydney.

Nielsen, P., 2009. *Coastal and estuarine processes*. Advanced series on ocean engineering. World Scientific, Singapore, 341 pp.

Nielsen, P., Brander, R., W. and Hughes, M., 2001. Rip currents: Observations of hydraulic gradients, friction factors and wave pump efficiency. *Coastal Dynamics '01*, Lund, Sweden, 483-492.

Nielsen, P., Guard, P.A., Callaghan, D.P. and Baldock, T.E., 2008. Observations of wave pump efficiency. *Coastal Engineering*, 55(1): 69-72.

Press, W.H., Teukolsku, S.A., Vetterling, W.T. and Flannery, B.P., 1992. *Numerical recipes in fortran: The art of scientific computing*. Cambridge University Press, Cambridge, 1486 pp.

Sugandika, T.A.N., Callaghan, D.P. and Nielsen, P., 2013. *Atoll lagoon flushing at manihiki in the cook islands*, Coasts and Ports, Sydney.

Vu, T.T.T., 2014. *Aspects of inlet geometry and dynamics*, The University of Queensland, 343 pp.

Wilson, B., 1997. *The efficiency of a wave pump*, University of Queensland, Brisbane, 46 pp.