3rd CADAM meeting - The Toce River test case : Numerical results analysis

S. Soares Frazão[1] and G. Testa[2]

[1]*FNRS research fellow*
Université catholique de Louvain – Génie Civil
Place du Levant 1, B-1348 Louvain-la-Neuve, Belgium
soares@gc.ucl.ac.be

[2]*Project Manager*
ENEL Polo Idraulico e Strutturale
Via G. Pozzobonelli 6, 20162 Milano, Italy
testa@pis.enel.it

Abstract

Within the EU Concerted Action on Dam break Modelling (CADAM) a series of blind simulations on dam-break flooding and associated flow phenomena was conducted. The Toce case takes place in a series of tests with progressive rise in difficulty, evolving from simplified geometries to a real valley case planned for the final meeting in November 99. The present case consists in an extreme flood event on a scale model and was simulated during the first six months of 1999. The model used is a 1:100 reproduction of the Italian alpine Toce valley.
The paper describes the experimental facility located at ENEL (Milano, Italy), the test cases and the numerical simulation performed by eight European Organisations. A general good and encouraging agreement between numerical results and experimental data was obtained, although several problems in modelling real topographies are outlined.

1) Introduction

The activity of the CADAM Concerted Action during the period between the Munich Workshop (8/9th October 1998) and the Milano Meeting (6/7th may 1999) was mainly devoted to the numerical simulation of an extreme flood event in the Toce River physical model.
The methodology adopted for the Toce was similar to the one used in the previous test cases: it consisted for the modellers in performing several "blind" numerical simulations followed by sensitivity tests. The test programme has been as follows:

- Toce model data available via FTP from 21st December 1998
- Numerical results returned via FTP by 13th April 1999
- Physical modelling results released to all participants who had submitted numerical results on 13th April 1999
- Sensitivity analysis performed by the modellers and presented at the meeting on 6th May 1999

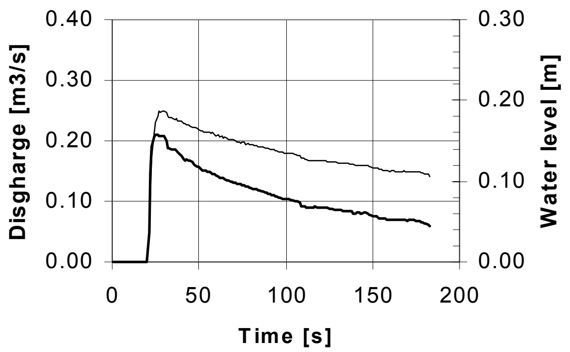
Figure 1 : Milano meeting participants and the Toce model

2) Description of the experimental facility

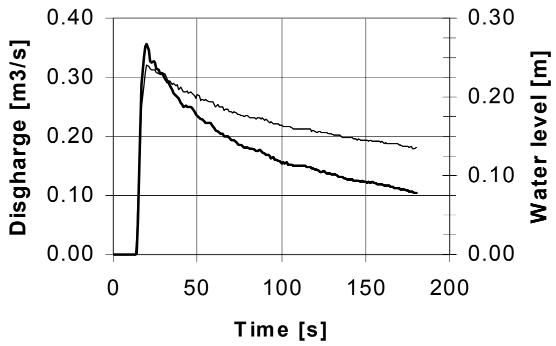
The physical model set up for the experimental tests is located at ENEL in Milano and reproduces a 5 km reach of the Toce River in the Northern Alps. Due to the morphology of the boundary cross-sections, critical flow occurs at the upstream and downstream ends of the reach, so that flow conditions outside the reach can be neglected. In order to simulate the valley, 72 cross-sections were surveyed; the same number of wooden shapes were then traced by a CAD-CAM process. After having put these shapes on a gravel layer, the valley was modelled with mortar. Bridges, barrages and villages were also reproduced on the model.
The scale is 1:100 and the total area of the experimental facility is 55x13 m. The model is located on a outdoor structure provided with tanks for the inflow and outflow and a computer controlled pumping circuit which can supply up to 0.5 m3/s of water. Water levels are measured by means of 28 gauges with pressure transducers.
Photographs of the scale models are shown on Figures 3, 4 and 5. The upstream part of the valley consists in large floodplains on both sides of the river. At mid distance between the upstream reservoir and the downstream end of the model, there is a lateral reservoir that can be overtopped during a catastrophic flooding. Further downstream, there is a small barrage on the river. The other main singularities in the model are the bridges and the buildings.

3) Flow in the scale model

Two hydrographs were selected to simulate an extreme flood event in the Toce valley (see Figure 2) : HY1 with a peak discharge of 0.2 m^3/s and HY2 with a peak discharge of 0.35 m^3/s. They are generated automatically by a pump. A rectangular tank is located at the upstream end of the facility, the tank is filled up by the pump and the water then flows into the valley. The model is initially dry, there is no initial flow in the river. The upstream inflow conditions were chosen in such a way that HY1 does not cause any overtopping of the reservoir banks (see the location of gauge P12, Figure 6), while it happens with the stronger HY2 where water fills up the reservoir and remains inside.



a) Hydrograph HY1



b) Hydrograph HY2

Figure 2 : Inflow conditions : Discharge (——) and water level (——) at the upstream boundary

During the experiment, water flows rapidly into the valley, inundating completely the large floodplains and submerging the bridges and buildings (see Figure 3 and Figure 4). The average slope is about 2%. Up to the intermediate reservoir, the front wave does not follow the meandering river axis (thalweg), it is almost 1D over the whole width of the valley. After the narrowing near the reservoir, the front wave follows the river axis while the water behind it inundates the floodplains. The small barrage (located near gague P21, see Figure 6) is completely overtopped. Due to the multiple reflections against the bridges or the buildings and within the valley, the flow develops a strong 2D structure. Figure 5 shows the overtopping of the lateral reservoir banks and the partial reflection of the water during the flood generated by HY2.

Figure 3 : Upstream part of the valley : dry river axis and flooding

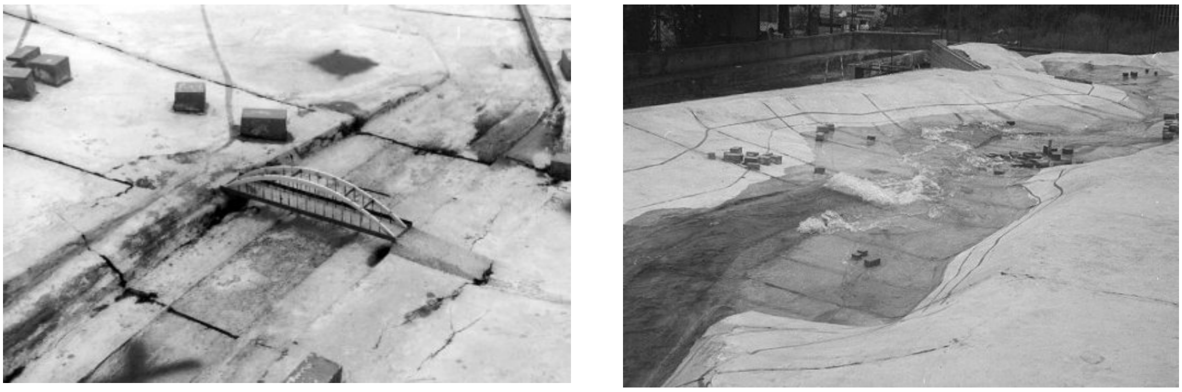


Figure 4 : Overtopped bridge and 3D jet effect on the flow

Figure 5 : Flood wave entering the reservoir

4) Tests performed by the modellers

a. Description of the test cases

Two different test cases were proposed to the modellers : one without overtopping of the reservoir (hydrograph HY1) and one with overtopping (hydrograph HY2). The modellers could use either a 1D or a 2D numerical model to compute the flow. The topographical data were given in the form of 65 cross-sections and a DTM about 140 000 (x,y,z) points on a square grid of 5 x 5 cm. The proposed test cases are summarised in the following table.

Test	Num. model type	Hydrograph
A	1D	HY1
B	1D	HY2
C	2D	HY1
D	2D	HY2

Table 1 : Summary of proposed test cases

A total of 32 water level gauges were placed along the scale model. Among those, a limited number was chosen to compare with the water levels predicted by the modellers. The selected gauges (see Figure 6) are P1, P4, P18, P21 and P26 for the 1D models and P1, P4, S6S, S6D, S8D, P9, P12, P18, P21 and P26 for the 2D models. For the 1D models, the gauges were selected on the main river axis. For the 2D model, additional gauges were chosen on the floodplains, and near singularities.

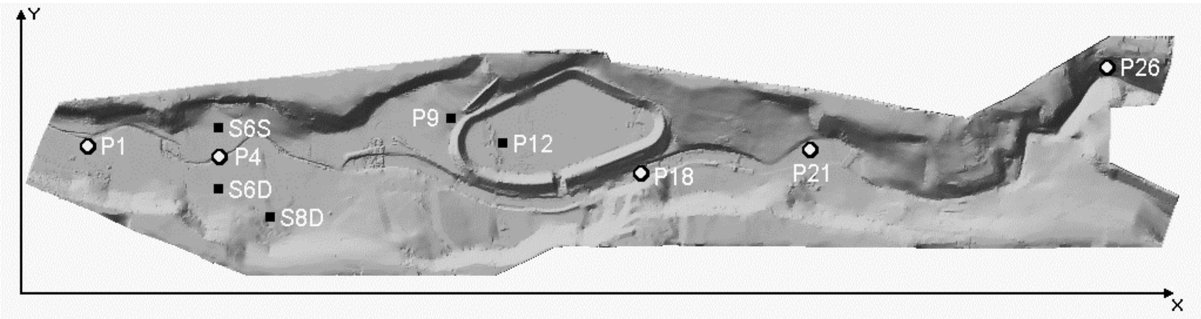


Figure 6 : Position of the gauging points (⭘ 1D and 2D, ■ 2D)

Table 2 indicates the names and organisations of the modellers who undertook the simulations, as well as the tests they performed. A total number of 22 results sets coming from 8 different modeller teams was sent to the organisers to compare with the measured water levels.

Name	Organisation	Test A	Test B	Test C	Test D
A. Paquier	CEMAGREF	X	X	X	X
N. Goutal	EDF	X		X	X
C. Rosu – M. Ahmed	HR Wallingford	X	X		
S. Soares	U.C. Louvain	X		X	X
M. Nujic	U.B.W. München			X	X
F. Alcrudo	U. T. Zaragoza			X	X
P. Brufau – I. Villanueva – P. Garcia-Navarro	U. T. Zaragoza	X	X	X	X
M. Szydlowski	T.U. Gdansk			X	X

Table 2 : List of participants and performed tests

b. Modelling techniques

All modellers work with the depth-averaged Saint-Venant shallow-water equations. Except HR Wallingford who uses the ISIS software based on the implicit finite-difference Preissmann scheme, all modellers compute the flow by an explicit finite-volume scheme. Most of them use the Roe solver to calculate the fluxes between adjacent cells (except S. Soares who uses a Boltzmann solver). The numerical models mainly differ in the way the friction and topographical source terms are computed.

Beside the differences in the numerical models themselves, we can point out the different ways the modellers used the available data. The exact shape of the buildings was given, however these were simulated by increasing the bottom level of the nearest nodes or by increasing locally the friction coefficient. Sometimes also, the buildings were not taken into account. The other singularities like the bridges and the downstream barrage were not always considered. The barrage was mostly represented like a weir, while the bridges were ignored by all modellers except A. Paquier, but with a negligible influence on the results. The bridges are in fact completely overtopped, leading to a separation of the flow and 3D features occur, which cannot be accurately represented by vertical averaged models.

The modellers had to construct their own mesh on the given (x,y,z) points, and this was done with variable level of refinement. Some of them built quadrangular meshes on the given topographical points, some others used triangular cells. The interesting mesh used by A. Paquier has to be pointed out, as it was built in such a way that the orientation of the quadrangles follows the main river axis. Doing this, the error induced by an arbitrary mesh on the direction of the flow propagation is minimised.

Another difference between the numerical models lies in the way the friction is introduced. An averaged measured Manning coefficient of 0.0162 $sm^{-1/3}$ was given for the physical model and was used by most of the modellers (sometimes transformed into a Strickler coefficient). Some modellers decided to increase locally the friction coefficient to represent the slowing effect of the buildings on the flow.

Table 3 and 4 summarise the main features of the computations run by the participants for the 1D and 2D test cases respectively.

Name	Cross sections	Friction	Buildings	Bridges	Barrage
Villanueva, UT Zaragoza	+/- 500	Manning 0.016	No	No	Yes
Goutal, EDF	200	Strickler 61	Yes : Strickler 15	No	Yes
Ahmed, HR Wallingford	200	Manning 0.02	No	No	Yes
Paquier, CEMAGREF	200	Strickler 60	No	1	Yes
Soares, UC Louvain	63	Manning 0.016	No	No	No

Table 3 : Main features of 1D computations

Name	Cells	Friction	Buildings	Bridges	Barrage
Villanueva, UT Zaragoza	dx = 0,05 m	Manning 0.016	No	No	Yes
Alcrudo, UT Zaragoza	dx = 0,15 m	Manning 0.02	Yes : Topography	No	Yes
Goutal, EDF	20 000	Strickler 61	Yes : Topography	No	Yes
Nujic, UBW München	dx = 0,05 m	Manning 0.016 Locally 0.02	Yes : Manning 0.045	No	Yes
Paquier, CEMAGREF	7 236	Strickler 60	No	1	Yes
Soares, UC Louvain	19 389	Manning 0.016	No	No	No
Szydlowski, TU Gdansk	dx = 0,2 m	Manning 0.016	No	No	?

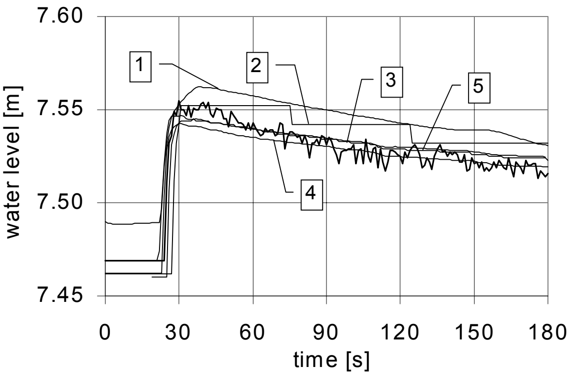
Table 4 : Main features of 2D computations

5) Comparison between numerical results and experimental measurements

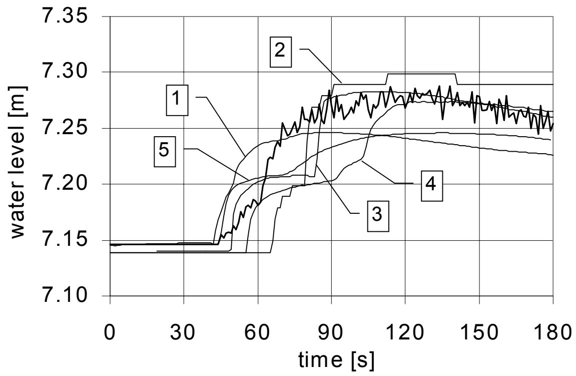
Generally speaking, the computed water levels are in acceptable agreement with the experimental data, and relatively close to each other. The plots in the following sections illustrate this and outline the problems encountered. The measured data shows an important number of oscillations which are not reproduced by the numerical models. Those oscillations come mainly from the free-surface which is not flat and shows a lot of small undulations and small waves. Some oscillations can also be due to the measurement devices themselves, as those consisted mostly in pressure gauges.

a. 1D models

Figure 7 shows the water level evolution at gauges P4 and P21. The numerical data come from 1D computations. Gauge P4 is located at the beginning of the valley, in the middle of the cross section. Despite the 2D nature of the flow, 1D models seem to compute a reasonably good averaged water level. Gauge P21 is located just before the barrage in the downstream part of the valley. The discrepancies between numerical models are more important and the agreement with the measured data is only approximately achieved once the flow has reached a quasi-permanent state.
It must be outlined that in the scale model, a longitudinal wall guides the water towards the downstream barrage, making the reflection stronger. This wall is not introduced in the numerical models and the barrage is simply represented by a weir.



Gauge P4



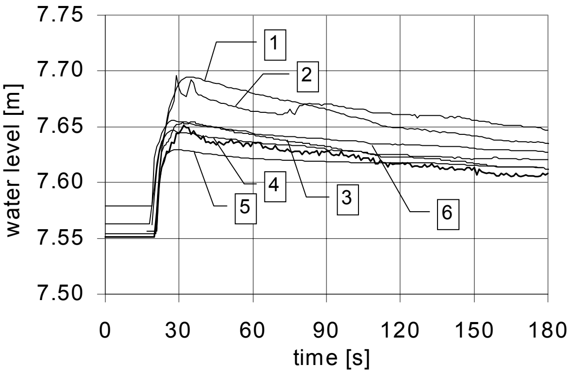
Gauge P21

Figure 7 : Comparison between experimental and numerical results for the 1D models : ——— experiment and —— numerical, 1:Ahmed (HR Wallingford), 2:Villanueva (UT Zaragoza), 3:Goutal (EDF), 4:Paquier (CEMAGREF), 5:Soares (UC Louvain)

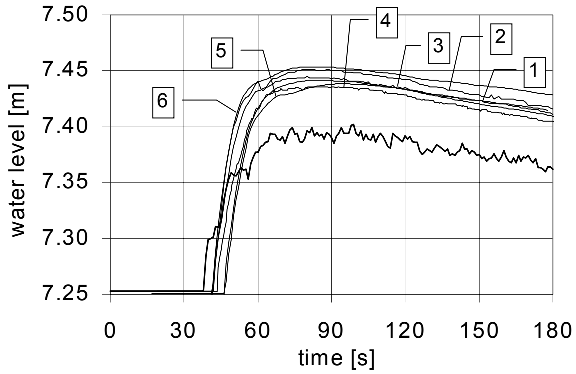
b. 2D models

Figure 8 shows a comparison between computed and experimental results at four different gauging points. Gauge P1 is located at the very beginning of the valley, and is thus directly affected by the way the inflow conditions are computed. The discharge and a measured water level in the tank (S1) were given to the modellers, as well as the water level in the first section of the valley (S2). Some modellers used this information to build a water level vs. discharge relation as inflow condition, which could be calibrated by the measured S2 values. Other modellers included the tank in the mesh, which allows to specify only the discharge as subcritical inflow condition. It appeared that the latter option gave a better representation of the upstream boundary condition. However, no significant consequence of the inflow condition choice could be noted on the results quality at the downstream gauges. For example, at gauge P18, all numerical models agree well together but disagree with the measures. A possible explanation is that the critical transition occurring in the narrowing is not well captured by the numerical models.

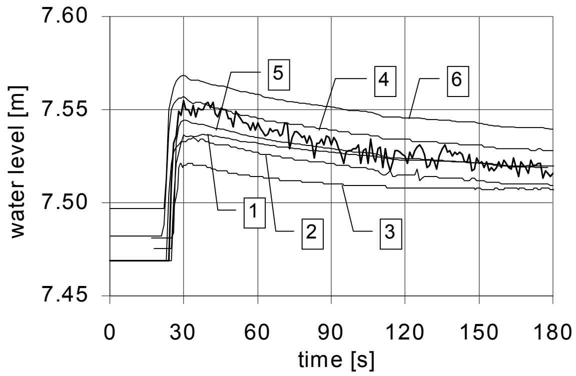
Gauges P4 and PS6D are located in the same cross section, in the upstream part of the valley. Most of the numerical results are close to the measures.



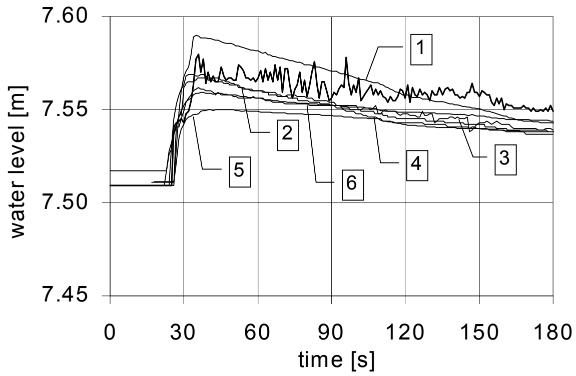
Gauge P1



Gauge P18



Gauge P4



Gauge S6D

Figure 8 : Comparison between experimental and numerical results for the 2D models : ▬▬ experiment and —— numerical, 1:Alcrudo (UT Zaragoza), 2:Goutal (EDF), 3:Nujic (UBW München), 4:Paquier (CEMAGREF), 5:Soares (UC Louvain), 6:Szydlowski (TU Gdansk)

c. Wave travel time between P1 and P26

After comparing the computed and measured water levels, an interesting point is to check if the predicted wave speed agrees with the reality of the water flowing down the scale model of the Toce valley. Figure 9 shows the front wave speed plotted in an *x-t* diagram, for both 1D and 2D computations. The experimental travel time was measured between gauges P1 and P26 and is of 40 s. Table 5 summarises the computed travel times. The results for the 1D computations are quite spread, ranging from 36 s to 79 s, and some are very close to the reality. The 2D computations show less spreading of the results, but give all too slow propagation time. The best estimated travel time is still 10 s too long, which represents a significant error. Up to now, no satisfactory explanation could be given for that wrong behaviour of 2D models.

1D models		2D models	
Name	Time	Name	Time
Ahmed, HR Wallingford	36 s	Alcrudo, UT Zaragoza	54 s
Villanueva, UT Zaragoza	79 s	Goutal, EDF	54 s
Goutal, EDF	42 s	Nujic, UBW München	54 s
Paquier, CEMAGREF	56 s	Paquier, CEMAGREF	51 s
Soares, UC Louvain	36 s	Soares, UC Louvain	58 s
		Szydlowski, TU Gdansk	50 s

Table 5 : Summary of travel times (experimental = 40s)

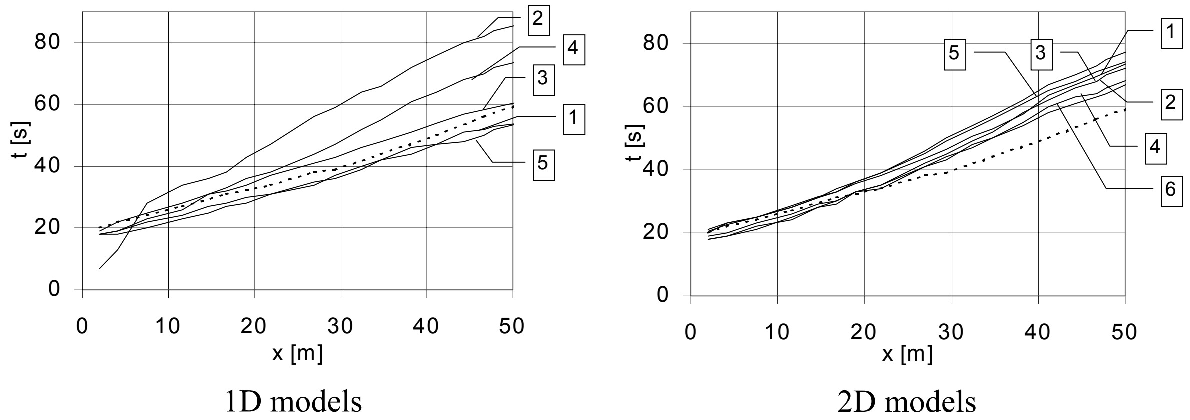


Figure 9 : Front propagation characteristics: ······ experiment and —— numerical for the 1D models : 1:Ahmed (HR Wallingford), 2:Villanueva (UT Zaragoza), 3:Goutal (EDF), 4:Paquier (CEMAGREF), 5:Soares (UC Louvain) and for the 2D models : 1:Alcrudo (UT Zaragoza), 2:Goutal (EDF), 3:Nujic (UBW München), 4:Paquier (CEMAGREF), 5:Soares (UC Louvain), 6:Szydlowski (TU Gdansk)

6) Conclusions

After former simulations devoted to idealised geometries consisting of channels with rectangular cross-sections, the Toce test was an important step towards real valley modelling. The main difference is perhaps the fact that here, the domain boundaries are not just defined by vertical walls, but are defined by the actual topography, leading to a series of "dry-wet" interfaces.

Despite this difficulty for the numerical models, a good agreement was found with the experiment, at least in terms of water levels. However, important discrepancies arose regarding the front wave travel time. It appeared that all 2D models computed a too slow wave, and until now, no satisfactory explanation of these delays could be given. Computations run on 1D models also showed a good agreement in terms of water levels, and were even better than the expected more accurate 2D models regarding the wave travel time. This, however, should not be taken as a general conclusion. Indeed, the Toce valley is rather wide, with large flooplains and it appeared that the flow presented a general 1D behaviour.

To really assess the performance of 1D and 2D numerical models, and define their range of application, more tests should be performed, on different types of valleys. This will be the aim of the next and last CADAM meeting.

7) Acknowledgements

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