NUMERICAL ANALYSIS AND THE REAL WORLD IT LOOKS PRETTY BUT IS IT RIGHT?

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Abstract - The routine usage of numerical tools such as finite element, finite difference and computational fluid dynamics analysis software in engineering design has increased in recent years. Advances in software and hardware technology mean more nonlinear, complex three-dimensional analyses are being performed. However, these powerful software, which are "black-box" in nature, may potentially lead to "computer-aided-disaster" in the hands of analysts who may have the "computing" skills but not necessary the extensive engineering experience. The strict implementation of quality assurance procedures may not necessary ensure the numerical model or the analysis technique is correct.

This paper describes how numerical analysis results can be validated in three real-world civil engineering applications with increasing complexity. These include a structural analysis of a steel water tank using finite element method, an investigation of flood-water flowing over a hydraulic structure using computational fluid dynamics method and simulation of a quay wall construction using finite difference method. The level of uncertainty of the input data and the trustworthiness of the computed results for each case will be discussed. Some interesting outcomes were uncovered during the analysis process.

The first case study showed that the quality of construction has a significant impact on the performance of the structure. However, the designer would probably not able to quantify and analyse such situation with any degree of accuracy during the design stage. The importance of monitoring at the end of construction is essential so that the measurement can form a datum for design verification as well as for future back-analysis if needed. Although the finite element analysis is a powerful numerical tool that can analyse complex problems, the analysts should still be prepared to come across unexpected outcome in situations where the behaviour of the problem may appear to be simple and well understood.

The rigorous validation process is highlighted in the second case study as computational fluid dynamic analysis was applied to an important spillway structure for the first time. It was done in a progressive manner starting with a 2D ogee spillway profile and working towards analysing the 3D model of the spillway in question. The computed results were compared with theoretical and physical test data at each stage. Despite the inherent nonlinear nature of fluid flow problems, the analysis was able to provide appropriate results for practical design purposes with confidence.

The final case study showed that the behaviour of the quay wall was influenced by the construction history and the way reclamation was carried out. The wall movement was correctly predicted using a simple nonlinear soil model, albeit qualitatively, despite the highly variable soil properties. The lack of continuous monitoring record made validation difficult.

The key to validating the computed results is to find an independent calculation that does not involve the use of numerical software tools. In many cases, these solutions are available. But in other cases, it can only resort to laboratory or field observations.

Introduction

Nowadays numerical analysis forms an integral part in most engineering design. The need for result validation is therefore vital throughout the design process so that the analysis technique/methodology can be trusted and designers have confidence in the computed results. The common practice is to validate the results against classical theory, experimental data, published data, performance of similar structures and numerical computation done by others. Sometimes benchmark or verification examples provided by the software developers may be used for this purpose - but they are rarely comprehensive enough to cover a full spectrum of problems. Before undertaking any numerical analysis, the analysts should decide how reliable the input data is, whether the software tool can solve the problem in question and how to validate the results. Although the validation process has been adopted as part of the quality assurance procedure by many practitioners, costly failures still happened [1].

Validation

The need for result validation can be reinforced by observing some bad industry practices in the use (abuse) of numerical analysis. Some engineers/analysts employed to carry out the numerical computation may not have a thorough understanding of the basic theory behind the computation, and/or they may not have enough practical engineering experience to deal with any hidden pitfalls.

Increasingly there is a tendency for engineering companies to employ draftspersons instead of graduate engineers to carry out numerical modelling and analysis just because some software has become so "CAD-like" and many claim to be easy to operate. Users would spend significant effort in creating a complex geometry model, meshing it with the appropriate elements, applying the boundary conditions (contacts, loads and fixities) for each load case, assigning properties and setting all the required flags/switches/buttons for submitting the analysis runs. Whilst some quality assurance procedures for self-checking could be followed during the pre-processing stage, by the time the computation was completed and the results were post-processed, many users would readily believe the output was correct – more or less.

Geometry creation is only part of the numerical modelling process. One of the most difficult problems is to deal with uncertainties throughout the whole design process. There are uncertainties associated with the input such as the material properties and loading sequence. There are uncertainties associated with the appropriateness of solution type, for example, whether the model would behave in a linear or nonlinear manner. And last but not least there are uncertainties associated with result interpretation.

There is no simple guide to good practice for analysts in making result validation and spotting problems in a numerical analysis. However, it can gradually be achieved by the following means:

- A good understanding of the numerical method process – this can be attained by formal education at undergraduate and/or postgraduate level, and enhanced further by self-study as part of one's continuous professional development.

- A good understanding of the basic theory and the range of solutions to a particular type of problem. Again this can be accomplished through education as above.

- A good experience in conducting numerical analysis as well as using engineering judgment in solving real-world problems. This can be gained through working in an environment where the analyst is properly supervised by experienced engineers.

It should be noted that the implementation of a quality assurance system is not a substitute for engineering judgment that lead to workable solutions. Very often it is necessary to carry out numerical "experiments" using a simple test model to fully understand the simulation technique and the fundamental behaviour of the problem before proceeding to analysing a large complex model. Experience has demonstrated that sometimes the test model itself might provide enough information for the analyst to arrive at the final design solution - the analysis of the corresponding large complex model is just to confirm design expectation.

The following case studies demonstrate how result validation was carried out and how the level of confidence and uncertainties were addressed.

Applications

In a typical civil engineering project numerical analysis may involve any one or a combination of the following three fundamental disciplines:

structural mechanics, geomechanics and fluid mechanics. The nature of the problem can be classified as one of soil-structure interaction, fluid-structure interaction, or soil-fluid interaction. In some instances it may involve all three. In view of the potential complexity, some assumptions and idealization have to be made to simplify the problem without losing much accuracy and still capturing the important behaviour for practical purpose. There are general-purpose as well as specialised numerical analysis software that can solve these problems. Both types of software have been used in the case studies.

Case 1 – Deflection of a steel water tank

A large circular steel water tank with a diameter of about 90m, at first filling, underwent large wall deflections that caused concerns in regard to long-term structural integrity of the tank. The height of water was approximately 10m at full storage capacity. The roof structure was almost entirely supported by columns located inside the tank. The strakes consisted of high-grade steel plates which were thicker in the bottom third of the wall. A primary wind girder was welded around the tank top whilst a secondary wind girder was located two-thirds above the base. The bottom strake was fillet welded to an annular base plate. The entire floor, except at the internal column's foundation, was covered by welded steel plates. The tank was founded on compacted fill overlying competent inter-layered sandstone and siltstone bedrock.

A series of axisymmetric finite element analyses (FEA) was performed to determine whether the observed deflections could have been predicted, and to calculate the stress state at the base of the wall because of the potential for fatigue failure under daily water filling and emptying. A three-dimensional model of a one-twelfth sector of the tank, which included the internal columns and roof beams, was analysed initially to investigate how much roof self-weight was supported by the wall and the validity of the axisymmetric assumption. The results of this analysis showed that the stiffness contribution from the roof structure was not significant and hence it was not included in the subsequent axisymmetric model. However, a small portion of the roof self-weight would be applied to the wall.

The axisymmetric model consisted of all the steel sections, fillet and butt welds and foundation (Figure 1). They were discretised with predominately 4-node incompatible mode quadrilateral with a few 3-node triangular axisymmetric elements. The welds were modelled such that load transfer was permitted through the weld material only. A fine mesh was utilized at the weld connection to accurately capture the stress state there. Roller supports were applied to the side and bottom boundaries of the model. The following loads were applied: self-weight of the steel structure, roof self-weight, hydrostatic pressure on the wall and uniform pressure on the floor corresponding to the water level. One model assumed the weld or the steel plates at the base was yielded to form a plastic hinge. A pin connection was modelled at the base of the wall for this case.

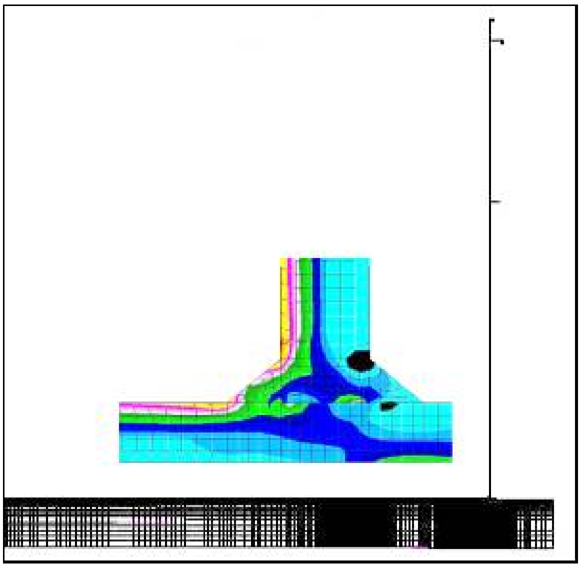

Figure 1 Partial FE mesh of tank/foundation. Insert shows mesh and stress distribution at wall base.

The wall deflections are shown in Figure 2. The range of measurement and the computed results are shown for comparison purposes. To validate the computed wall deflections, the classical theory based on Timoshenko and Woinowsky-Krieger [2] for the two wall thicknesses were also plotted in the figure. It can be observed that the computed deflection is bounded by the theoretical calculations. The transition due to the change in wall thickness was captured in the analysis. This provided confidence in the finite element model. The influence of the wind girders and the base restrained can also be seen. It is possible the installation of the wind girders could have introduced some initial strain causing the wall top to pull inward at the end of construction. Hoop action dominated the wall behaviour except close to the base where bending action occurred.

Even though the computed maximum deflection was of the same order as those measured, the height at which the maximum bulging occurred was not predicted. In fact, the survey data suggested some possible scenarios: formation of a plastic hinge at the base (but the computed stresses in this region had not exceeded the yield strength); localized bearing failure of the subgrade material (again no obvious tell-tell sign such as cracks was seen on site); or there was some built-in geometric imperfection at the end of tank construction. Back-analysis was performed on a pre-deformed tank such that the as-measured deflection was “recovered” under the hydrostatic water load. However, the computed stresses were well in excess of yield. Unfortunately the tank was not surveyed immediately after completion prior to first filling.

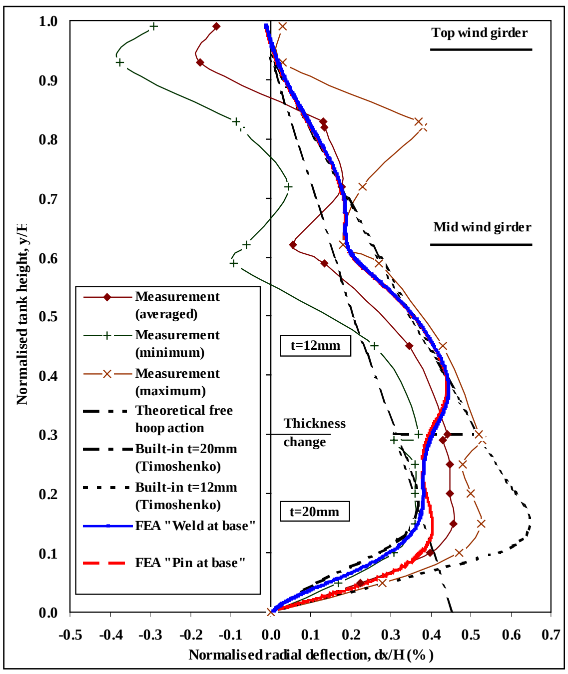

Figure 2 Wall deflection of water tank

It is interesting to note that the original design and construction of the tank was carried out in the early 2000s. It used the relevant standard [3] in the design calculations. The standard assumes the tank wall acts under hoop action alone and ignores the constraint at the base that is clearly not the case for this structure. The magnitude of wall deflection could have been determined either by classical theory [2], analytical method developed by Rish [4] who took into account the foundation stiffness or numerical analysis such as FEA. The use of high-grade steel may warrant designer to chose a thinner section that would be adequate for strength but not necessary for serviceability – bending stiffness is governed by the cube of thickness. The subsequent wall deflection profile under water load would be influenced by the quality of workmanship. This would have been difficult to estimate during the design stage.

Case 2 – Spillway discharge

Many dam structures in Australia were designed and built in the 1950s and 60s with limited hydrological information. As such existing spillway structures are under-sized to cope with the revised probable maximum flood levels. Potential problems such as the generation of negative pressure over spillway crest under increased flood condition would be encountered. This may cause instability or cavitation damage to the spillway and gate structures. Historically, scaled physical models have been constructed in hydraulic laboratories to study these behaviours, but they are expensive, time-consuming and there are many difficulties associated with scaling effects. Today, with the use of high-performance computers and more efficient computational fluid dynamics (CFD) codes, the behaviour of hydraulic structures can be investigated numerically in a reasonable time and expense.

As this analysis technique was applied for the first time in Australia on the largest concrete gravity dam, which provides a major source of water supply to a large metropolitan area, the need to carry out validation was essential. This was incorporated in the investigation process as shown in Figure 3. The flowchart illustrates how it progressed from a simple 2D to the detailed 3D spillway model.

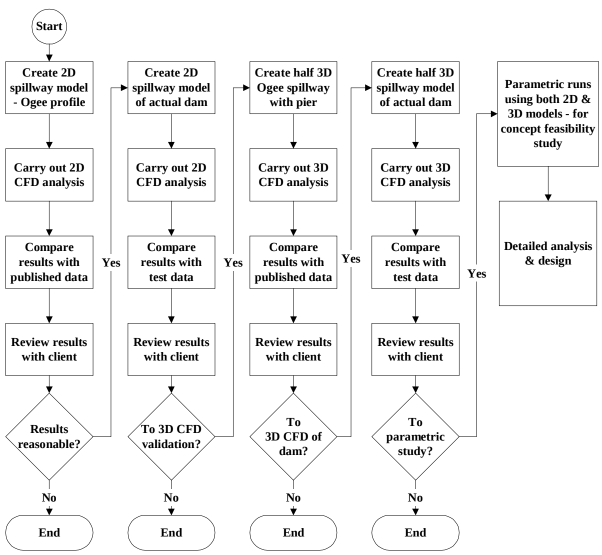

Figure 3 Flowchart showing the validation process

An ogee spillway profile (see Figure 4) was chosen for validation because there are extensive data published by the US Army Corps of Engineers [5]. The computed results were reviewed at each stage of the investigation. Should they deviate significantly from the published data the project would be abandoned. This has been mutually agreed with the client before the project commenced.

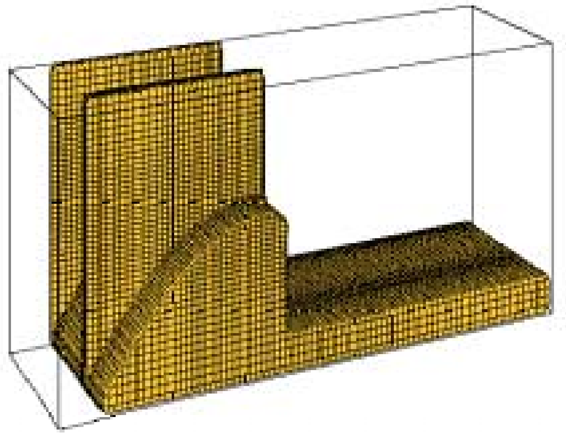

Figure 4 A view of the ogee spillway and Type 2 piers in the 3D CFD model

One of the earlier difficulties of this kind of analysis was the accurate computation of the free surface in open-channel gravity flow problems. The use of adaptive meshing and iterative method in tracking the free surface was employed in some finite volume CFD codes but with limited success. The code used for the present study solves the Navier-Stokes equation by the SOLA-VOF method. Finite difference method was used to solve for the transient behaviour of the fluid motion. The volume of fluid (VOF) function is used for computing free surface motion [6]. Details of the analysis are described elsewhere [7].

The computed crest pressure distribution, free surface profile and discharge rate at steady-state were used for validation purposes. The pressure distributions along the spillway crest under different upstream heads (H) are shown in Figure 5. Some pressure oscillations may probably attribute to the way the code handles the computation at the interface between the regular mesh and the curved spillway obstacle. A much finer mesh would have smoothed out some of these irregularities. The influence of the piers on the pressure distribution was correctly predicted in the 3D model (Figure 6).

The computed free surface profiles (Figure 7) were also in good agreement with the published data. A similar validation exercise was conducted by Savage and Johnson [8] using the same CFD code that further provide confidence in the analysis technique. Subsequent analysis of the spillway in question gave reasonably good results when compared with those obtained from the scaled physical model tests.

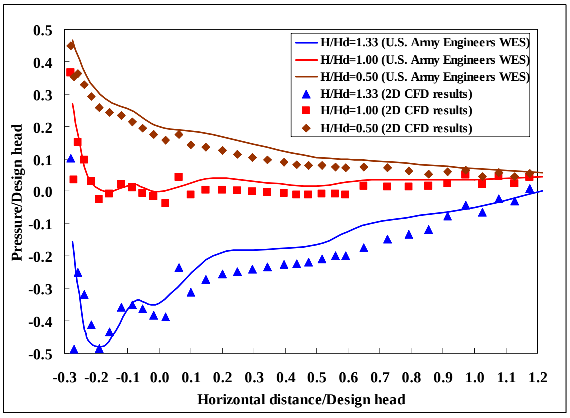

Figure 5 Comparison of crest pressure for various heads (2D model), H_d is the design head

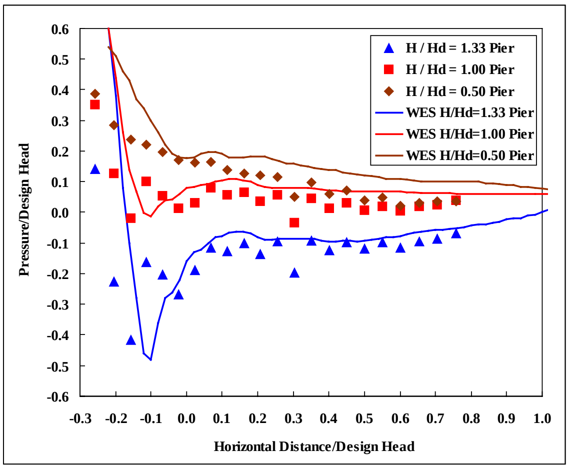

Figure 6 Comparison of crest pressure next to pier (3D model)

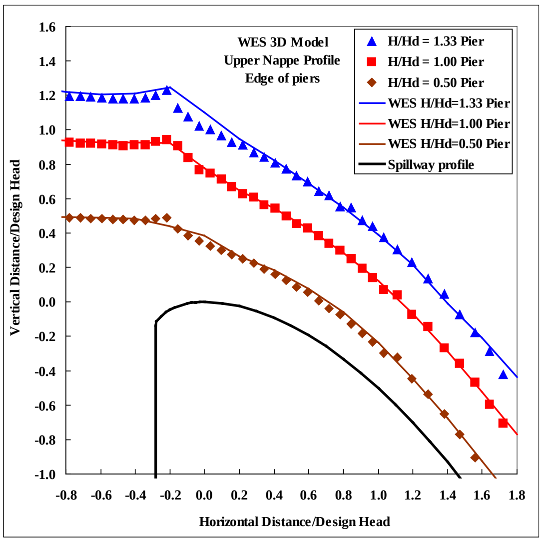

Figure 7 Upper nappe profile next to pier

In the analysis the geometry of the spillway and the water properties were well defined. The water was assumed to be incompressible and has constant properties at a fixed temperature. The spillway boundary was assumed to be smooth because good quality concrete surface finish could be achieved in practice. The uncertainties would come from two sources: the mesh density and the selection of an appropriate turbulence model. The mesh size is constrained by the amount of memory and the clock speed of the computer. Although the turbulent flow at high Reynolds number can be computed with a very fine mesh that can even capture the formation of vortices and eddies, the current mesh density was sufficiently fine to predict the required variables for validation and design purposes. The investigation also showed the result was not significantly influenced by the choice of turbulence model such as the large eddy, k-ε and RNG models. Obviously, the introduction of wall roughness and turbulence model would reduce the discharge rate. But again the analysis results show that they have little impact for the mesh used. Future analysis would investigate the discretisation error caused by different mesh density.

Case 3 – Quay wall construction

A major container port facility was built some 25 years ago having minimal numerical analysis conducted during the design phase. The use of such analysis tool was deemed not cost-effective back then. Extensive dredging and reclamation work was carried out for the construction of a 2km long quay wall that has a number of container cranes running along side. Since the completion of the facility the quay wall, which consists of a series of concrete counterfort units grouted together, and the rear crane beam have been moving continuously to the extent that re-leveling work was done to the rear beam so that the cranes can operate normally. However, a more permanent solution was sought to arrest the movement of both affected structures. An explicit finite difference analysis that can handle soil-structure interaction and construction simulation was used to help with the ranking of different remedial options.

Before embarking on analysing various proposed remediation such as grouted columns, tieback anchors and piled supports, it was decided that the computational model ought to be calibrated against observations so that selection of the soil and structural properties as well as the construction process was appropriate.

The geology and geotechnical information was assessed from site investigation reports containing in-situ and laboratory test data. A considerable scatter of test data is expected for any given soil type encountered on site given the coverage of the facility. Some typical records of the standard penetration test (SPT) blow counts (N) and cone penetration test (CPT) resistance (q_c) for the hydraulic sand fills are shown in Figures 8 and 9.

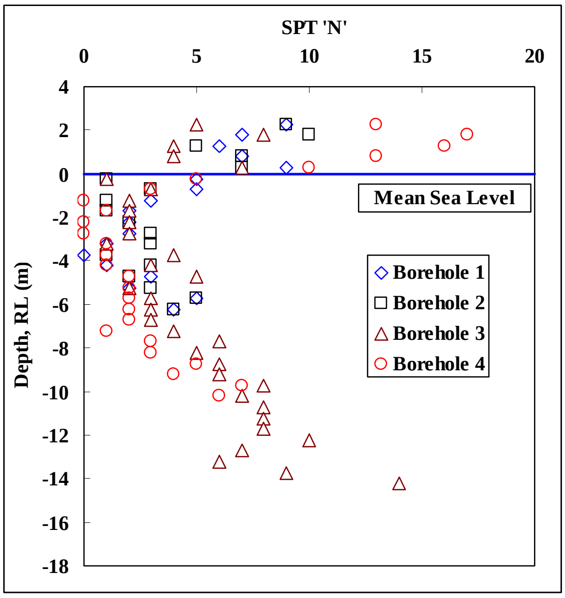

Figure 8 SPT 'N' profiles

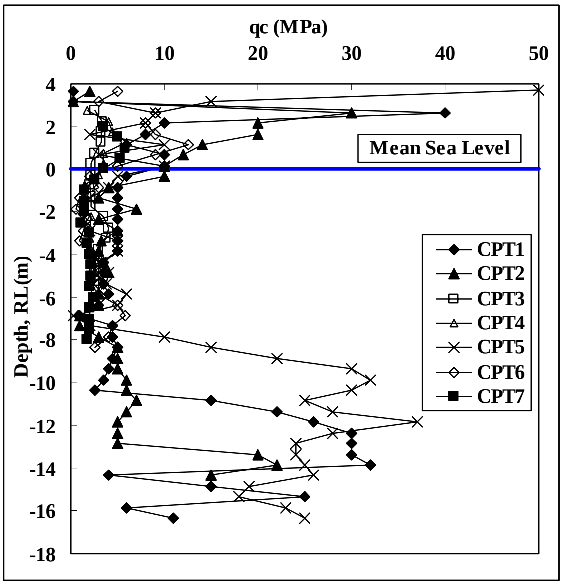

Figure 9 CPT profiles

The contrast in strength and stiffness of the sand fill above and below the mean sea level can be observed from these results. It had been suggested [9] that this phenomenon was attributed to the placement method. There was also variability in the properties for the vibro-compacted sand at the foundation level. The soil properties selected for the analysis were based on test data, experience from nearby site and published data of similar soil conditions. They are summarised in Table 1. In general the construction sequence of the facility is as follows:

1. Removal of pockets of soft marine clay by dredging
2. Dredging of sand to the required level
3. Vibro-compaction of the sand on which the counterfort units were to be founded
4. Placement of gravel for the quay wall foundation.
5. Placement of concrete counterfort units weighing 360 tonne each
6. Placement of hydraulic sand fill behind the units
7. Surcharging the fill just behind the capping beam
8. Construct capping beam and place more sand fill to the finished level
9. Additional surcharge prior to the operation of container cranes.

Table 1 Soil properties used in the construction simulation of the quay wall

Properties Units	**E'** MPa	**υ'** -	**γ** kN/m^3	**c'** kPa	**ϕ'** deg	**ψ** deg
Gravel bed	35	0.2	20.0	0	40	0
Vibro-compacted sand	27.8	0.3	19.3	0	37	0
Native sand	80	0.3	20.0	0	40	0
Fissured clay	48	0.2	18.0	5	15	0
Clay	160	0.2	18.0	0	24	0
Hydraulic sand fill above mean sea level	30	0.3	17.0	0	32	0
Hydraulic sand fill below mean sea level	25	0.3	20.0	0	28	0

E' – elastic modulus, υ' – Poisson's ratio, γ - total unit weight, c' – cohesion, ϕ' – friction angle, ψ - dilation angle.

In the numerical simulation of the 2D plane strain model the construction sequence (Figure 10) and loading was simplified/idealised to the following steps:

1. The starting condition of the seabed consisted of the vibrocompacted sand, gravel bed, native sand, clay and fissured clay at depth. The "in-situ" stresses were also switched on in this step.
2. Placement of counterfort unit (using equivalent linear elastic beam elements) with a vertical force applied through the centre of gravity of the unit to represent the buoyant self-weight.
3. Sequentially placing hydraulic sand fill behind the unit to the level prior to surcharging.
4. Apply an equivalent trapezoidal pressure to represent the surcharge.
5. Placement of capping beam and the sand fill to the required level.
6. Apply additional surcharge.
7. Application of repeated loads from the crane seaward and landward legs.

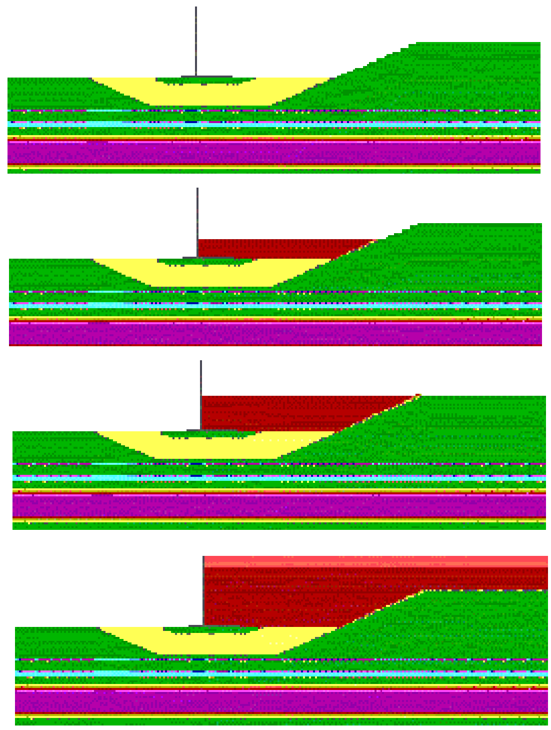

Figure 10 Construction sequence

In the analysis, the appropriate densities were used to represent the materials that were submerged and those that were above the mean sea level. Since long-term movement of the quay wall was of interest, drained soil parameters were used. The soil was assumed to obey the Mohr-Coulomb failure criteria in the analysis. A simple elastic-perfectly plastic stress-strain behaviour was assumed.

The history of the quay wall movement as represented by a series of rigid body diagrams, is shown in Figure 11. The computed vertical and horizontal movements at the top and base of the wall are shown in Figures 12 and 13. Also, plotted in the figures were the monitored data and their upper and lower limits (bounded in the appropriate boxes). Despite the amount of scatter in the measurement, the computed movements for the wall construction compared reasonably well. It should be noted that there was no attempt to vary the soil properties in the analysis to match the predictions with the survey data.

The ratcheting effect of the repeated crane loads can be observed. Unfortunately there was no datum for the wall movement under repetitive crane loads and hence these predicted movements could not be compared. In view of the complexity of the problem and the highly variable soil properties, the computed results are remarkably encouraging.

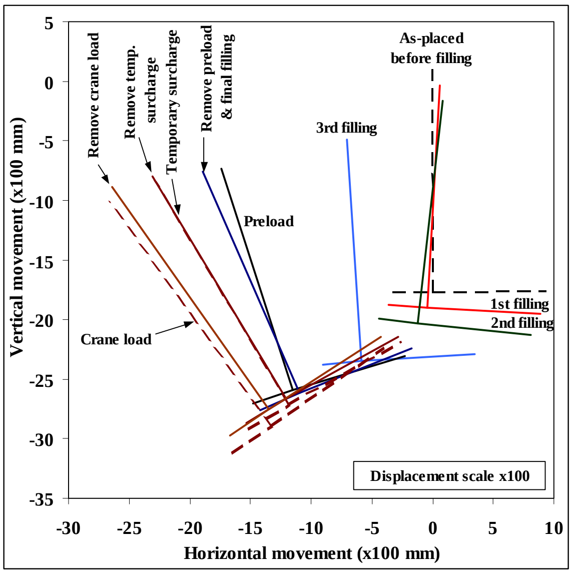

Figure 11 Wall deformations

The development of plastic zones in the soil was also computed from the analysis. It was found that the soil below the toe of the wall was over-stressed on a number of occasions. The contact pressures were used to determine an indicative factor of safety (FOS) against bearing failure due to incline loading. It has been reported that the bearing capacity was strongly influenced by the method of calculation [10]. As the original foundation design was based on the Danish code [11] it was used in this case for consistency. The development of FOS as function of eccentricity and the ratio of horizontal to vertical thrust (H/V) are shown in Figures 14 and 15 respectively.

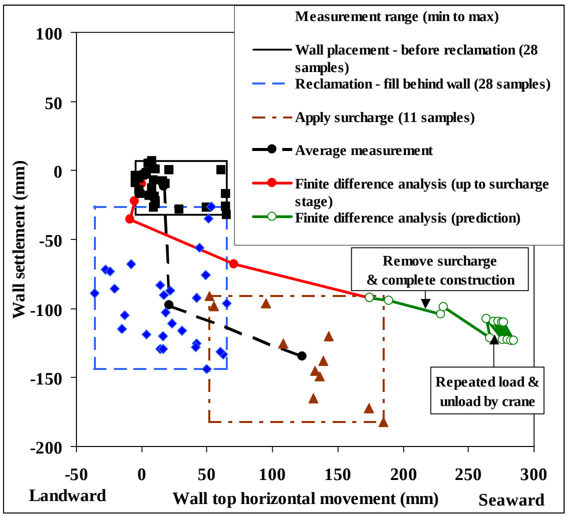

Figure 12 Wall top movements

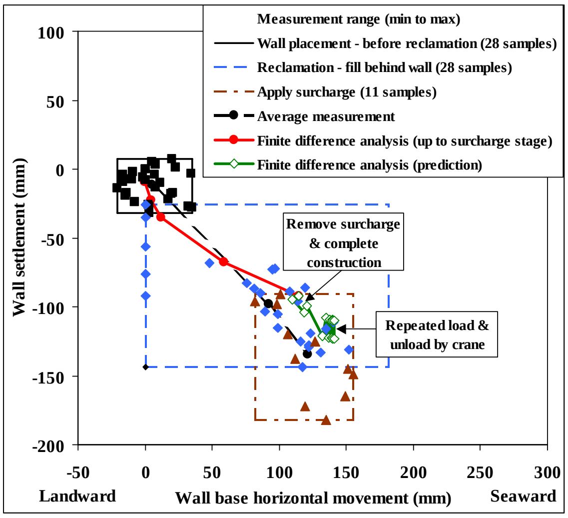

Figure 13 Wall base movements

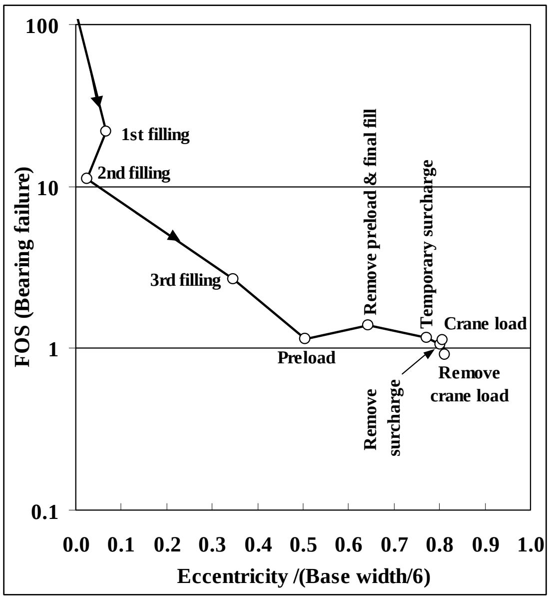

Figure 14 'FOS' vs. eccentricity

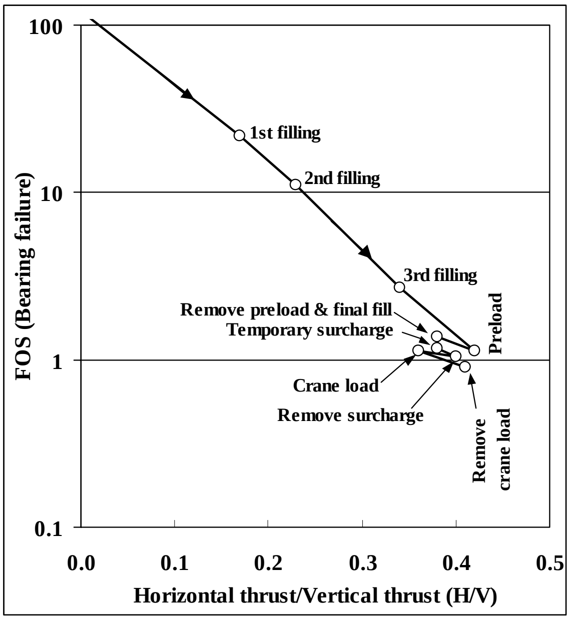

Figure 15 'FOS' vs. H/V ratio

The figures show the wall was close to localised bearing failure during surcharge and repeated loading stages. The apparent increase in FOS under the crane load was due to the reduction in eccentricity as the vertical load on the wall increases while the horizontal pressure from the retained soil remaining more or less constant.

Concluding Remarks

The validation process of three very different real-world applications has been described. The main features and findings of each case are summarised in Table 2. The material and loading uncertainties as well as the result expectation are highlighted. It was found that the quality of construction has a significant impact on the structure's performance – something that the analysts may not be able to quantify and accurately analyse during the design phase of a project. The importance of monitoring immediately after the structure was completed should not be overlooked. This will form a useful datum for future back-analysis. Despite the fact that the numerical tools could analyse these complex problems, the analysts should still be prepared to identify which parameters are or are not important. In analysing an unfamiliar problem, the validation process should be done incrementally. Perhaps a key to finding a validation method is to ask if there are other ways to arrive at the solution without the use of numerical analysis tools. In many cases, these solutions exist after extensive literature search. But in other cases, laboratory tests and field observations would be the only alterative.

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Table 2 Summary of findings for the three case studies

	Case 1: Deflection of a steel water tank	**Case 2: Spillway discharge**	**Case 3: Quay wall construction**
Governing mechanics:	Structural	Fluid	Soil
Other interactions:	Soil (support stiffness) Fluid (pressure)	Structural (obstacle/flow boundary)	Structural (concrete wall)
Numerical analysis type:	Finite element stress/deformation	Computational fluid dynamics (finite difference transient dynamic method)	Explicit finite difference
Analysis code:	ANSYS V5.7 (ANSYS, Inc.)	FLOW-3D V7.7 (Flow Science)	FLAC V4.0 (Itasca)
Dimension:	Axisymmetric	2D and 3D half symmetry	2D plane strain
Mesh density:	Very fine – in the order of fraction of a millimeter in the region of interest	Fairly fine – edge length of cell ranges from 0.4 to 3.0m	Fine to coarse – grid length ranges from 0.125m (area of interest) to over 1m (close to far field boundaries)
Nonlinearity:	Small deflection linear elastic (for most cases) Large deformation (one case) Boundary condition (contact between plates – all cases)	Solving the momentum equations (inherently nonlinear)	Material (elastic-perfectly plastic stress-strain assumption)
Material properties uncertainties:	Well defined for structural steel and weld material. Range of soil stiffness used for sensitivity study (to bound the solutions)	Well defined for incompressible water at constant temperature	Highly variable for geo-materials. Fairly well defined for concrete structure
Material model:	Linear elastic – well defined for steel Linear elastic – foundation material	Laminar (most cases) Turbulence model (k-ε) – for sensitivity study	Mohr-Coulomb yield criteria (simple soil model)
Pressure/loads uncertainties:	Well defined except for geometric imperfection/locked-in stress during construction	Upstream head is well defined	Self-weights of fill and wall are well defined Variability in fill density and stiffness is likely
Complexity of analysis:	Not complex	Fairly complex	Complex
Initial expectation of prediction:	Very good agreement expected	Fairly reasonable agreement expected – probably depends on mesh density	Not optimistic due to high uncertainties in material properties and construction sequence
Validation method used (with indicative dates):	Classical theory (1950s) Published paper (1977)	Published data (guidelines since 1950s) and limited physical test data on scaled model (early 1990s)	Limited measured data during installation (1970s)
Variable(s) used for comparison:	Wall deflections	Pressure distributions, free surface profile and discharge rates	Vertical and horizontal movements at top and base of quay wall
How did the prediction compare?	Similar order of magnitude but did not predict the correct maximum location	Reasonably good for the mesh size used	Reasonably good given the variability of the geo-materials' properties and construction history
Other desirable information for the validation exercise	Detailed construction records and initial survey	Information on the accuracy of measured data in the physical scaled model	More complete records of survey data and datum measurements for crane loads.