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Implications of the 2011 Great East Japan Tsunami on sea defence design

Alison Raby [a,*], Joshua Macabuag [b], Antonios Pomonis [c], Sean Wilkinson [d], Tiziana Rossetto [b]

[a] School of Marine Science and Engineering, Plymouth University, Plymouth, UK
[b] EPICentre, Department of Civil, Environmental & Geomatic Engineering, University College London, UK
[c] Director, Cambridge Architectural Research Ltd., UK
[d] School of Civil Engineering and Geosciences, Newcastle University, Newcastle upon Tyne, UK

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ABSTRACT

After the 2004 Boxing Day tsunami, much of the world's effort to defend against tsunami concentrated on tsunami warning and evacuation. The 2011 Great East Japan Earthquake and Tsunami led to direct and indirect losses as well as the deaths of many vulnerable members of Japan's coastal communities. This event has resulted in Japan rethinking and revising its design codes for sea defence structures. The new guidance emerging from this process is a valuable resource for other countries re-evaluating their own current mitigation strategies and this paper presents details of this process. The paper starts with the history of sea defence design standards in Japan and explains the process of revision of design guidelines since 2011. Examples of sea defences that failed and have since been rebuilt, observed during the two Earthquake Engineering Field Investigation Team (EEFIT) missions of 2011 and 2013, are also presented. The paper concludes with a discussion of international approaches and their application to nuclear power stations in Japan and the UK.

1. Introduction

In 2011 Japan was considered to be the best prepared nation on earth to withstand a large tsunami attack on its coasts, with sea defence structures (breakwaters, sea dikes and seawalls) specifically designed to afford sufficient protection to coastal settlements and critical infrastructure. Massive detached breakwaters were built in bays to defend great industrial ports and their populations; sea dikes were constructed along much of the coastal plain areas to protect low-lying agricultural land and towns from both tsunamis and storm surges; and seawalls, some of which were 10 m or more in height, were built as a result of previous tsunamis to provide protection for busy settlements. However, the size of the waves generated by the unexpectedly large magnitude of the 2011 Great East Japan Earthquake (Moment Magnitude 9) led to sea defences and other coastal structures being overwhelmed and in many cases completely or partially destroyed. Overtopping of the sea protection wall at the Fukushima Daiichi nuclear power station led to the loss of seawater pump facilities for the reactor cooling water causing a major release of radioactive material.

Due to the non-structural measures in place along Japan's vulnerable coastlines *i.e.* comprehensive warning systems and well-rehearsed evacuation plans, casualty figures were relatively low in comparison to the levels of devastation caused by the tsunami. However, this paper focuses on the structural measures to defend against tsunamis (though not the seismic considerations, see *e.g.* [46]) and charts the evolution of Japanese design guidelines and standards in the light of this catastrophic event. It describes the research that has been conducted into the failure mechanisms of key sea defence structures, explains the different levels to which sea defence structures must now be built and describes the detailed disaster scenario document that now exists for design considerations. Photographs and observations from two Earthquake Engineering Field Investigation Team (EEFIT) missions to Japan are included to illustrate the damage caused to defences and the reconstruction that has subsequently taken place in some locations. The paper then presents an overview of international approaches to sea defence design, where they exist, including the new American Society of Civil Engineers standard. It concludes by providing details of post-2011 sea defence structures built to protect an existing Japanese nuclear power station at Hamaoka, and the new Hinkley Point C power station in the UK.

* Corresponding author.
E-mail addresses: alison.raby@plymouth.ac.uk (A. Raby), joshua.macabuag.12@ucl.ac.uk (J. Macabuag), antpomon@ath.forthnet.gr (A. Pomonis), Sean.Wilkinson@newcastle.ac.uk (S. Wilkinson), t.rossetto@ucl.ac.uk (T. Rossetto).

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Notation

The following symbols are used in this paper:

a_I	incident tsunami height;
g	gravitational acceleration;
p_u	uplift pressure;
p_1	maximum value of pressure;
η^*	effective tsunami height; and
ρ_0	density of seawater.

2. Japanese Tsunami Design Procedures pre-2011

Before discussing Japanese guidance the terminology used to define such structures is given in Fig. 1. The underlined terminology is that which is used in this paper.

Japan has a rich history of guidance available to coastal engineers for the design of sea defence structures. In addition to the internationally renowned Yoshimi Goda who produced three editions of his book on the design of such structures over a nearly 30 year period [24] there are official *Technical Standards* that have been established under government acts. These prescribe the required technical criteria that should be applied in the construction, renovation or maintenance of facilities [65]. *Manuals* are also published as reference documents that provide detailed design guidance. The standards and manuals available for ports and harbour facilities prior to 2011 are shown in Fig. 2 and explained as follows.

The Technical Standards comprise a brief summary of each standard with an accompanying commentary which is divided into written notes on the standard followed by reference information with pertinent equations [65]; this information may be in the form of academic articles, design manuals, technical notes *etc*. The standards are prepared by bureaus within the respective ministries. Revisions to standards are introduced when there have been major changes *e.g.* the Technical Standards for Port and Harbour Facilities, moved from designing for regular to irregular waves in 1978 or the incorporation of performance-based design in 2007 (following pressure from the Technical Barriers to Trade Agreement [51]).

The production of the 2007 edition of the Port and Harbour Facilities technical standards was a collaboration involving:

- the Ports and Harbours Bureau (PHB) of the Ministry of Land, Infrastructure, Transport and Tourism (MLIT);
- the National Institute for Land and Infrastructure Management (NILIM) of the MLIT; and
- the Port and Airport Research Institute (PARI).

An English translation of these 2007 technical standards was released by the Overseas Coastal Area Development Institute of Japan in 2009; this translation will be the document referred to in this paper as PHB [51].

The last Design Manual was released in 2000 [7]. This was prepared by the Committee on Coastal Engineering of the Japanese Society of Civil Engineering, a neutral body that had representation across different Ministries and from academics and practicing engineers (Mizuguchi and Iwata, 1999).

The design process in Japan might typically involve a national

Terminology	Schematic diagram of structure section
Breakwater (generic)	
Coastal levee [33] / Dike [38] / Reveted embankment [59] / Revetment [59] / Sea dike [5]	
Breast wall [38] / Seawall [53]	

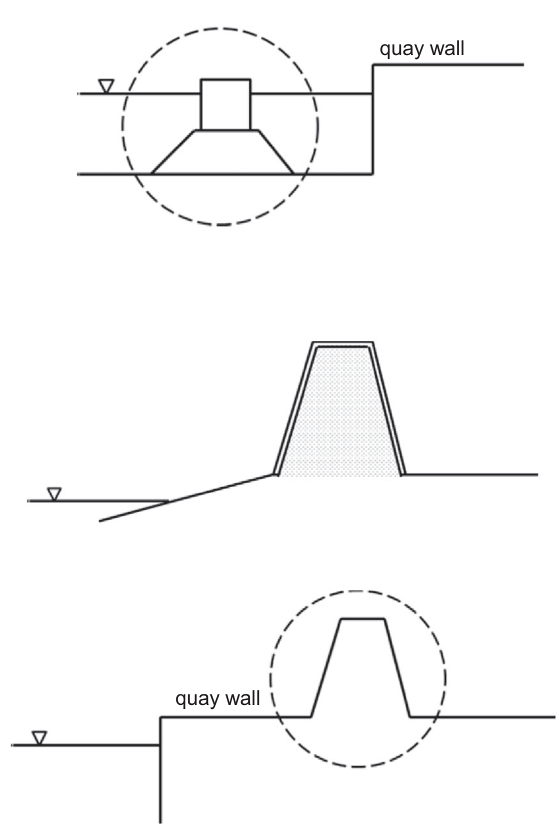

Fig. 1. Sea defence structure terminology [5,33,38,53,59].

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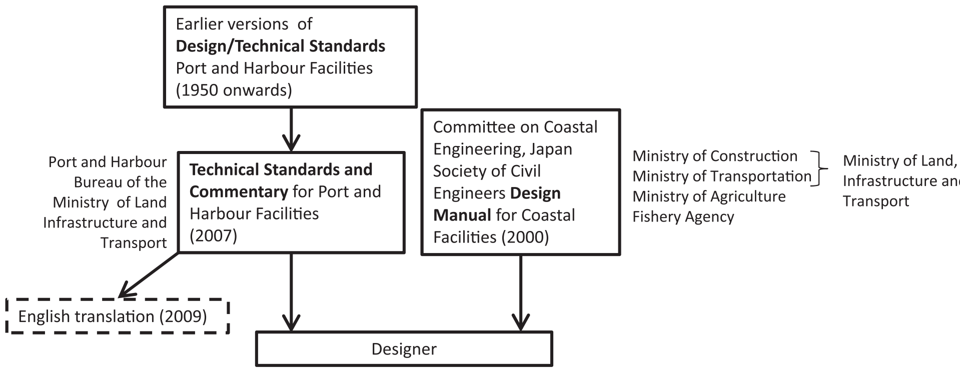

Fig. 2. Official Japanese guidance available prior to 2011.

or local government design office, perhaps with assistance from engineers in the private sector. The designs are then examined according to the standards by the relevant Bureau. Prefectures generally use ministry Standards and Manuals rather than developing their own and budgets from the ministries, prefectures, and big cities like Kobe and Yokohama (which have their own harbours), far outweigh those of the private sectors [34].

The 2007 Technical Standard for Port and Harbour Facilities [51] contains extensive guidance on the design of sea defence structures subject to tsunami. On the size of the tsunami, the standard requires that this should be determined by either numerical analysis or on the basis of historical tsunami, with the larger value selected [62]. This approach is in sharp contrast to what was done in the 20th century where structural design had apparently been based upon tsunami magnitudes experienced in the preceding decades.

As an example of the detail that is included in the standards, Fig. 3 illustrates the theoretical tsunami force and distribution of pressures on a caisson breakwater. These breakwaters are sometimes built specifically for defending a port from tsunami.

The effective height used for the calculation of pressures is given by

$$\eta^* = 3.0a_I \tag{1}$$

where η^* is the effective tsunami height and a_I is the incident tsunami height.

Pressure values increase linearly as shown in Fig. 3 to a maximum value of

$$p_I = 2.2\rho_0 g a_I \tag{2}$$

where $\rho_0 g$ is the unit weight of seawater.

The uplift pressure at the base of the front surface is given by

$$p_u = p_I \tag{3}$$

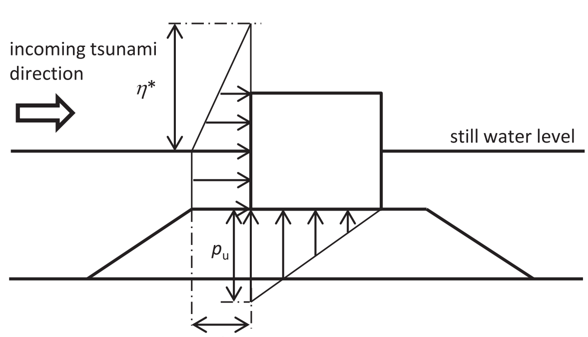

Fig. 3. Tsunami wave load distribution on vertical wall based upon Fig. 5.2 of PHB [51].

The performance verification of tsunami breakwaters requires an examination of stability against sliding and overturning of the upright section, and failure due to insufficient bearing capacity of the foundation. Consideration must also be given to the effect of the tsunami on water levels, recommending that numerical simulations are undertaken to evaluate the water level inside and outside of the breakwater, warning that the inside water level may not be the still water level. Further, it recommends the use of hydraulic model tests to deduce the tsunami force as the current theoretical understanding is not adequate.

For sea dikes it is necessary to look at the Technical Standards and Commentaries of Coastal Protection Facilities (though there is no English translation). Regarding seawalls, those structures which are required to defend against tsunami should have dimensions appropriate to perform this function according to the 2007 Technical Standards [51]. They are also to retain their structural stability even if the function of the seawall, *e.g.* ability to resist overtopping, is exceeded.

3. Field observations following the 2011 tsunami

The first of the two EEFIT missions to Japan took place 11 weeks after the 11 March 2011 earthquake and tsunami, following repair of the train line through the affected region. Within the team various civil engineering disciplines were represented including coastal engineers. Japan's reputation for coastal engineering is unrivalled: between 1965 and 1985 they constructed more than 1300 breakwaters [60] and they had also provided training to tsunami-prone countries following the 2004 Boxing Day tsunami under the auspices of UNESCO (ICHARM) [58]. It was therefore important to visit the country to see how the structures had performed. Visual observations of a variety of sea defence structures were undertaken and comprehensive details are provided in the EEFIT reports [17,18] and by Fraser et al. [23]. Fig. 4 indicates the key observation locations in the Tohoku region.

Some 8500 m of breakwaters failed during the 2011 event [53] including dedicated tsunami breakwaters like the World record breakwater at Kamaishi. Whilst barely visible from the town its destruction is worth reporting here. Comprising two sections, one of 670 m length and the other 770 m length it was built in a maximum water depth of 63 m. A cross-section is shown in Fig. 5. The failure of the breakwater has been subject to physical and numerical modelling investigations carried out by the Port and Airport Research Institute (PARI). They concluded that the structure failed due to a combination of two effects caused by the tsunami overflow: water level difference between the two sides of the breakwater causing an increase in the lateral force and the overflow scouring causing a reduction in friction between the caisson and mound leading to sliding [1].

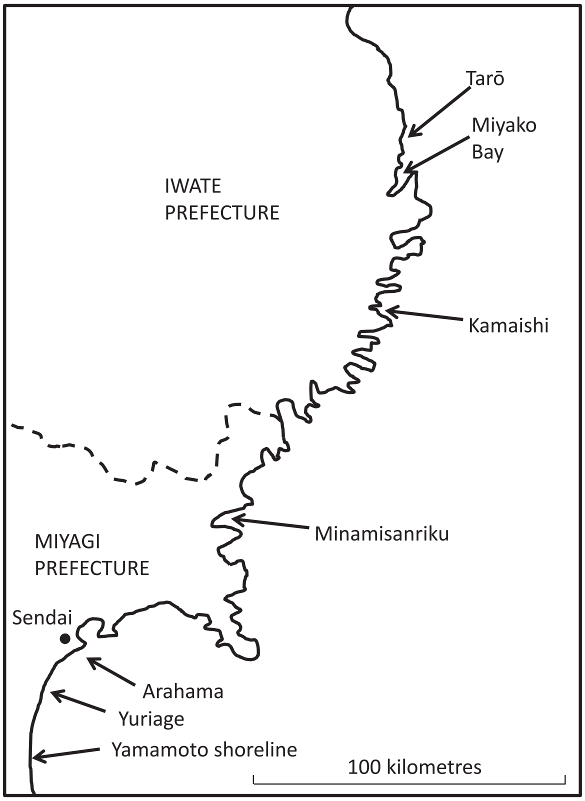

Fig. 4. Outline map of the EEFIT observation region, indicating locations of key coastal structures.

Fig. 6 shows a variety of dislocated concrete elements photographed in the bay at Tarō, amongst which were parts of a breakwater, as evidenced by the inclined harbour light.

The sea dikes shown in Fig. 7 showed clear effects of scour on the lee side leading to cracking of insufficiently-reinforced concrete lattice work.

Seawall failures of both gravity seawalls and compacted sand core structures were also observed (Fig. 8). Failure mechanisms of the Tarō seawall included the removal of rear slope protection blocks due to the tsunami overflow, the collapse of the crest parapet due to impulsive fluid forces, the collapse of the front protection blocks due to draw-down and there was also evidence of shear failure between blocks [29].

4. Updates to Japanese Tsunami Design Procedures post-2011

4.1. Tsunami classifications

One of the key criticisms of the sea defence structures was that

Fig. 6. Concrete debris from breakwater and seawalls offshore of Tarō, captured during the 2011 EEFIT mission.

they were not designed for a tsunami of the magnitude experienced in 2011 [3]. Design water levels were based upon tsunamis from fairly recent history *e.g*. the Meiji Sanriku tsunami of 1896, which caused a significant number of casualties (around 22,000 deaths) but had smaller recorded run-up levels than the 2011 tsunami [19]. The issue of inadequate design water levels has been addressed following negotiations between the government and disaster management experts post-2011 and Japan now has a two level description of tsunami hazard [61]. The levels are defined purely according to their return period at a particular location. Level 1 tsunamis are those that occur every '50–60 to 150–160 years' and Level 2 events occurring every few hundred to few thousand years [61]. Estimations of the inundation level are obtained from a combination of historic tsunami levels and numerical modelling of past and potential future tsunamigenic earthquakes. It has been decided that all sea defences should prevent inundation against a Level 1 event. However, whilst they should resist immediate structural failure for larger events [2] they should not be designed to stop overtopping as it would not be economically feasible. Incidentally, PIANC (World Association for Waterborne Transport Infrastructure) have a working group addressing Mitigation of Tsunami Disasters in Ports and they advise that reinforcements that would enable structures to withstand Level 2 events are an important measure in tsunami resilience: if a structure can survive, albeit in a damaged state, it will still afford a level of protection [54]. However, despite the hard engineering solutions that might be implemented, Level 2 events principally require *non-structural* measures to ensure life-safety *e.g.* warnings and evacuation procedures that were found to be effective in the 2011 event [17]. The latter measures also play an essential role for protection against Level 1 events in what is described as "Integrated Protection" *i.e.* both structural and non-structural measures for tsunami-resilient cities [36].

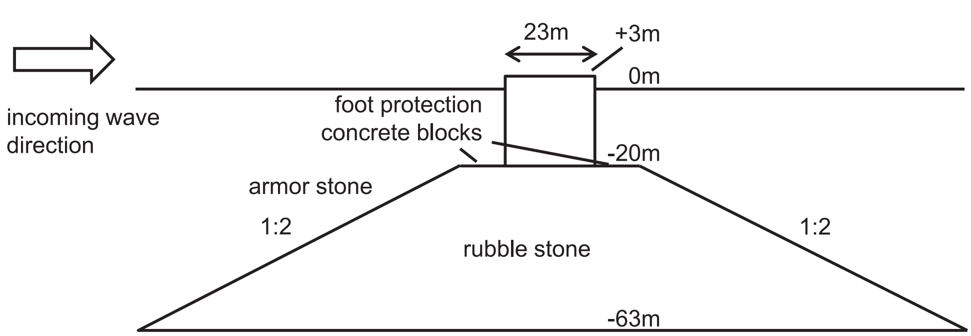

Fig. 5. Cross-section through the Kamaishi South breakwater (based on Ref. [40]).

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Fig. 7. Damage to Yamamoto sea dikes observed during the 2011 EEFIT mission: (a) exposed sand core on the lee side; (b) evidence of sand core having been washed out; (c) exposed section through the sea dike; and (d) remnants of the dike into the distance, lying in pools of seawater to the lee side of the structure having not subsequently drained away.

4.2. Revision of codes

Following the 2011 event and after the human recovery phase there was a great deal of activity by coastal engineers in site surveying and conducting laboratory and numerical modelling of the failed coastal structures. Some work was carried out within government organisations and some by researchers in universities. Much work was undertaken on failure mechanisms as reported in PIANC [54] and research literature (*e.g.* [1,33,31]).

Outputs from these efforts have been reflected in revisions to the technical standards, manuals and guidelines. As mentioned earlier, these exist to inform Japanese engineers of the most appropriate design for a facility. Fig. 9 shows the evolution of these documents following the 2011 event.

As mentioned previously, prior to 2011 there existed Technical Standards prepared by the Port and Harbour Bureau of the Ministry of Land, Infrastructure and Transport [51] and the Committee on Coastal Engineering Design Manual [7]. Following the 2011 event the publication: *A Draught Manual for Developing Earthquake-Tsunami Disaster Scenarios including Damage to Public Works* [43] was produced as an 'urgent action' by NILIM for MLIT, collecting together knowledge of tsunami forces.

In parallel with this, the Technical Standards document of 2007 was revised in 2014 [52] and incorporates new guidelines including the 2013 *Guidelines for tsunami-resistant breakwaters*, produced by the PHB, NILIM and PARI. Various official guidelines have been issued based upon research activity and these have fed into the revised technical standards. At this time there is no official English translation of these standards.

Incidentally, the 2000 version of the design manual [7] is now out of print though is still used by engineers and researchers as it describes the theoretical reasoning in some detail and was the original authoritative text for the design parameters [34].

In the following sections aspects of the guidelines, technical standards, design and prediction manuals pertinent to design for tsunami defence will be discussed. Many of the texts are still only available in Japanese; what is presented in the following are translations obtained as part of the 2013 EEFIT mission.

4.2.1. General requirements

In addition to considering tsunami loads which the structure must withstand, it is necessary to take into account the earthquake loads that may have preceded the waves. Effects of both subsidence and liquefaction must also be considered because they may lead to a reduction in the crest height of the structure. It is the crest height that determines the extent to which the structure can resist overtopping [43]. All sea defence structures must be able to withstand Level 1 seismic motion that typically occurs once or twice in its lifetime, but those that defend critical infrastructure need to be designed with larger and less frequent Level 2 events in mind.

Tsunami loads on structures are to be estimated from

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Fig. 8. Seawall damage observed during the 2011 EEFIT Mission: (a) damage to seawall and quay at Minamisanriku; (b) close-up of seawall block at Minamisanriku showing no interlinkages; (c) recovery work on the seawall at Miyako Bay; and (d) remaining seawall buttress at Tarō.

simulations undertaken by the Central Disaster Prevention Council and local authorities. If simulations are not possible other appropriate methods (undisclosed) are to be used [43].

The NILIM disaster scenario manual [43] recognises that a tsunami may consist of a number of waves where subsequent waves could be larger than the first wave. Therefore, tsunami damage to the sea defence facility may reduce its inundation prevention capability, and may lead to increased damage to the land behind the defence due to subsequent tsunami waves. A wide range of suggested wave pressure distribution sources are provided in a detailed diagram and in tabular format. The vast majority of the scientific research that forms the basis is post-2000. The following considerations were made:

- the location of the structure with respect to the shoreline;
- the orientation of the structure;
- whether soliton breakup is likely *i.e.* for steep bathymetries the tsunami waves will split into several short-period waves; and
- whether the wave imparts an impact force to the structure in addition to the hydro-static load.

The NILIM disaster scenario manual [43] also suggests equations for the load on top of the structure and on its lee side and for the return flow loads.

MLIT [37] compares the 2007 and 2014 technical standards and suggests that the pressure coefficient of Eq. (2) is to be increased from 2.2 to 3.0 when the tsunami is a bore-type (where the leading edge of the tsunami has broken) *i.e.* a 36% increase in predicted force. It should be noted that this factor is still less than the value of 3.5 given by Ikeno et al. [27] for gently sloping seabeds where soliton breakup may occur. Finally, there are other conceptual changes in the revised document, referring to tsunamis in two categories: expected or exceptionally large scale.

4.2.2. *Breakwaters*

The NILIM disaster scenario manual [43] gives a comprehensive flow chart illustrating the mechanisms by which a composite breakwater may fail; this is translated and reproduced in Fig. 10.

The flow chart covers the full range breakwater failure mechanisms from the initial effect of the tsunami (*e.g.* caisson damage due to debris impact and scouring of foundation), the effect of these initial issues on the positioning of the structure (*e.g.* tilting or subsidence) right through to the hydrodynamics (*e.g.* increase in flow speed or wave pressure convergence) and ultimately the failure of the structure. The manual also gives sources of equations to calculate pressures and necessary masses of armouring stones such as tetrapods. It is noted that it is not straightforward to calculate the effect of a tsunami on tetrapods.

The Port and Airport Research Institute (PARI) contributed to a document entitled *Guideline for Tsunami-Resistant Design of Breakwaters (a proposal)* by MLIT, presenting laboratory tests undertaken in 2012. Photographs of results from hydraulic tests with different angles of overflow are given in MLIT [39]. The key requirements are to redirect the flow of the overtopping tsunami so

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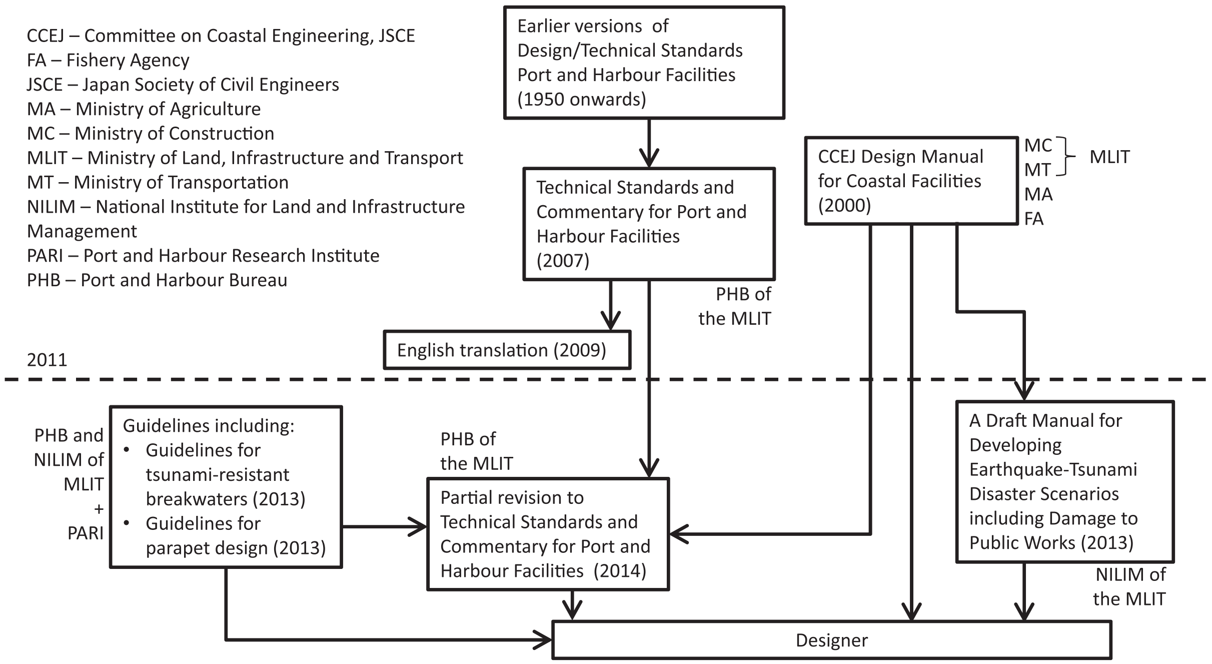

Fig. 9. Evolution of official Japanese design guidelines following the 2011 tsunami.

that it avoids scour on the lee side and to provide much more extensive protective armouring on the lee side. Fig. 11 shows original and revised designs for a caisson breakwater based on a NILIM newsletter [44].

As reported in the *Ministry of Education, Culture, Sports, Science and Technology in Japan White Paper* [35], by March 2016 MLIT is committed to completing all 'breakwater recovery work' that will see the revised crown shapes and lee side armour protection. Repair of the Kamaishi breakwater was already underway during the [18] mission.

4.2.3. Sea dikes

The NILIM disaster scenario manual [43] provides a tsunami loading diagram for sea dikes (Fig. 12). These particular illustrations, which relate to overturning as a design consideration, do not address scour on the lee side though it is described elsewhere in the document.

Following the circulation of prevention measures suggested by the *Committee on Tsunami Protection Measures at Coastal Areas*, Suwa et al. [64] indicate that investigative hydraulic experiments have been undertaken. These have been reported in a NILIM Technical Flash [42] and published by the Japan Society for Civil Engineers [32]. These have led to the following important guidance for the design of sea dikes:

- protect the landward toe of the structure from scour by providing improved foundations of up to 5m wide and 2m deep as shown in the NILIM document describing the experiments [45];
- avoid unevenness on the landward side of the dike by using interlocked blocks with notches at their top and bottom edges;
- avoid any single block of the dike being subject to negative pressure by interlocking all units (including the crown).

These recommendations have been incorporated into the rebuilt Sendai Bay dikes [64], immediately adjacent to the international airport. The dike is flanked with 2 tonne interconnected concrete blocks [28]. The [18] mission visited this area and photographs of the completed dikes near Yuriage are shown in Fig. 13. At a meeting with the Vice-Mayor of Iwanuma City during the 2013 EEFIT mission, he expressed the view that these sea defences would not last more than 50 years, but the inland barrier of a string of man-made hills would endure. These so-called Millennium Hope Hills are an example of several lines of defence that are being implemented; others include raised roadways with inland lock gates [36].

Fig. 14 shows both newly completed and under construction sea dikes at Arahama. The crest elevation was approximately 2 m higher than the pre-tsunami dike, and is assumed to be at the standard 7.2 m above Tokyo Peil (T.P., the fundamental metric datum of Japan), providing protection against Level 1 events, experienced every few decades. The dike armour comprised tessellated pre-cast concrete units. All the elements, even the steps, were of pre-cast construction. The sections under construction revealed the rubble/soil core topped by a geotextile membrane, a new recommendation.

Due to the depth of sand at the time it was not clear from the Yuriage and Arahama sites the extent of landward toe protection. Details of the armour units at both locations showed variations (differently shaped concrete blocks and at Arahama the lifting holes were grouted whereas at Yuriage they remained unfilled); the EEFIT team was informed by Mr Shinichi Endo, Director of Earthquake Disasters of MLIT's Tōhoku office, that whilst the designs were slightly different the contractors used the same specifications.

4.2.4. Seawalls

According to the NILIM disaster scenario manual [43] tsunami damage to the seawall can be split into three classifications: wave force damage, frontal scouring pattern and rear scouring pattern. Mention is particularly made of scour due to drawdown of the tsunami. A flow chart, translated and reproduced in Fig. 15 shows the damage chain to which gravity-based seawall structures are subjected, with the succession of damage resulting from the three classifications.

One of the very largest seawalls, in the town of Tarō, failed spectacularly in places (see Figs. 8(d) and 6), though not at the oldest sections completed in 1958 which had also proved effective in the 1960 Chilean tsunami [17]. Ironically the town was

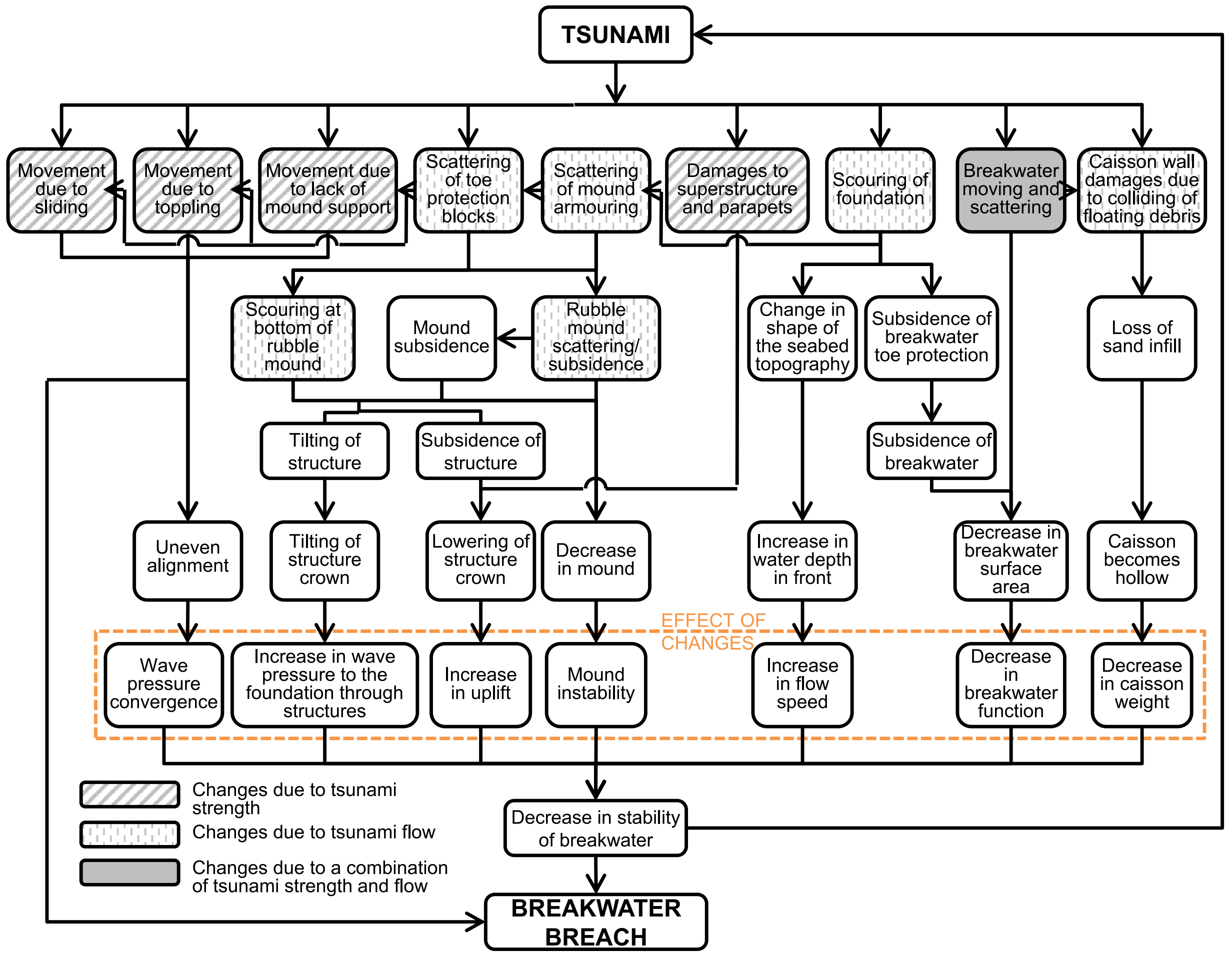

Fig. 10. Correlation of chains of main tsunami damage in caisson composite breakwaters (based on [43]).

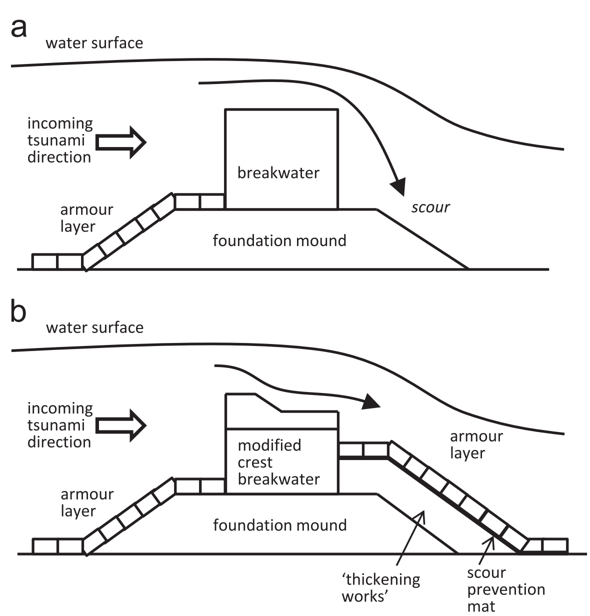

Fig. 11. Caisson breakwater section (a) former design (b) recommended design for a 'strong breakwater' able to resist a tsunami beyond the design height (based upon [44]).

nicknamed Tsunami-Tarō because of how it had survived the earlier disaster due to its protective seawall. Due to the enormous extent of the destruction and the rapid clearance of this particular site at the time of the 2011 EEFIT mission, it was not possible to identify which of the three classifications was dominant though Ishikawa et al. [29] present very detailed observations and both numerical and physical modelling results for the case of this seawall collapse, providing a number of suggested collapse mechanisms as mentioned in Section 3. The replacement sections of the wall will crucially have steel reinforcement [30], notably absent from the previous failed sections (see Fig. 8). During the 2013 EEFIT mission the port of Minamisanriku was re-visited to see progress on seawall reconstruction, but no change was observed in the damaged structures (see Fig. 16) though the fishing market had been rebuilt.

5. Design approaches to sea defence structures

5.1. International design experience

Early Japanese design approaches evolved independently of the international community due to general cultural isolation and the language barrier. Horikawa [26] estimated that only around 5% to 15% of Japanese Coastal Engineering papers are published in English. Some of the earliest work on wave loading was conducted by Hiroi in 1919 and was still in use until Goda produced his work in the second half of the century [26,65]. During the middle part of the 20th Century, Japan formalised their design guidelines, incorporating some US guidance produced by the US Army Corps of Engineers in their *Shore Protection, Planning and Design* guide, an

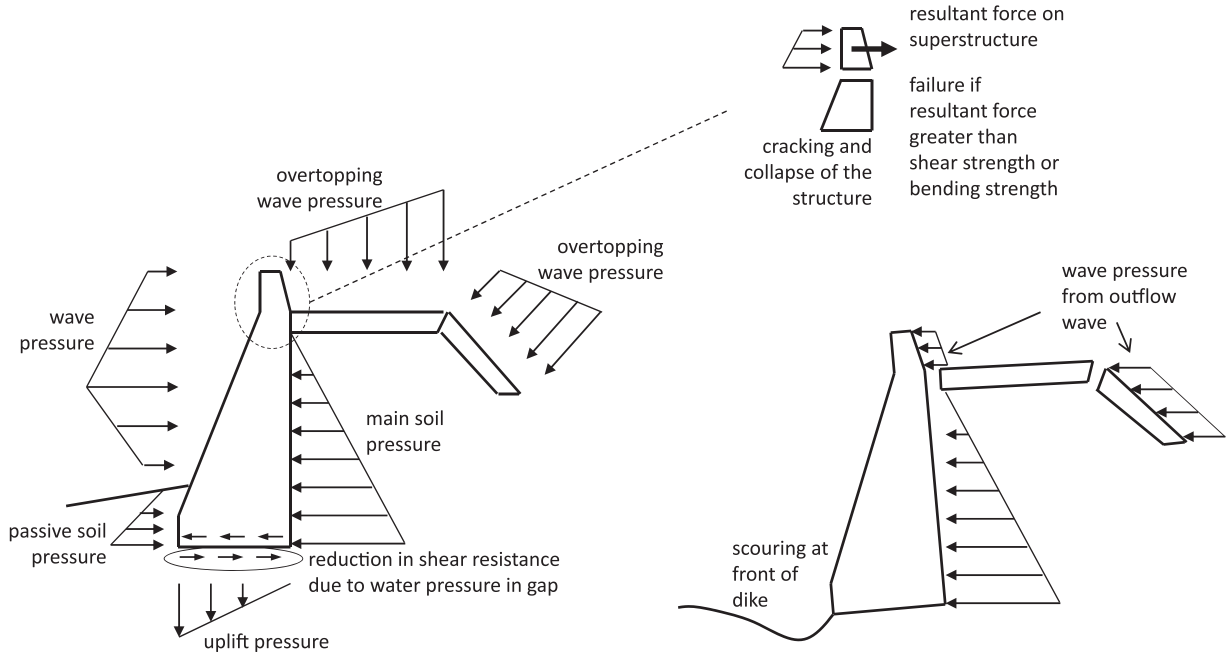

Fig. 12. Loading diagram of sea dikes (translated from [43]).

early precursor to the Coastal Engineering Manual [5]. These days, far more of the Japanese design guidelines are available to non-Japanese speakers, with the comprehensive technical standards being officially translated into English within a short time of publication, as indicated in Fig. 2. Yamamoto and Fukute [67] commented on the need to harmonise Japanese standards with international standards. This has evidently been achieved as the 2007 Technical Standards are now consistent with ISO 21650 *Actions from waves and currents on coastal structures* as well as international standards on reliability of structures and seismic actions for geotechnical works [51].

Interestingly whilst ISO 21650 contains descriptive information about tsunamis (generally alongside information on storm surges), it does not provide any design methods or equations for determining loads specifically due to tsunamis. According to the Norwegian Geotechnical Institute [41] there was some discussion about the inclusion of action from tsunamis, but at that time it was deemed that there were insufficient research results available for inclusion in the standard. It does recommend that the tsunami characteristics are predicted based on past or numerical modelling of potential future tsunamis. There is proposed provision for inclusion of basic tsunami considerations in the next revision of ISO/CD 3010 (Basis for Design of Structures), though this primarily pertains to onshore structures and so will not be addressed here.

Pilarczyk [56] makes mention of the fact that countries around the world are developing their own standards in isolation, having

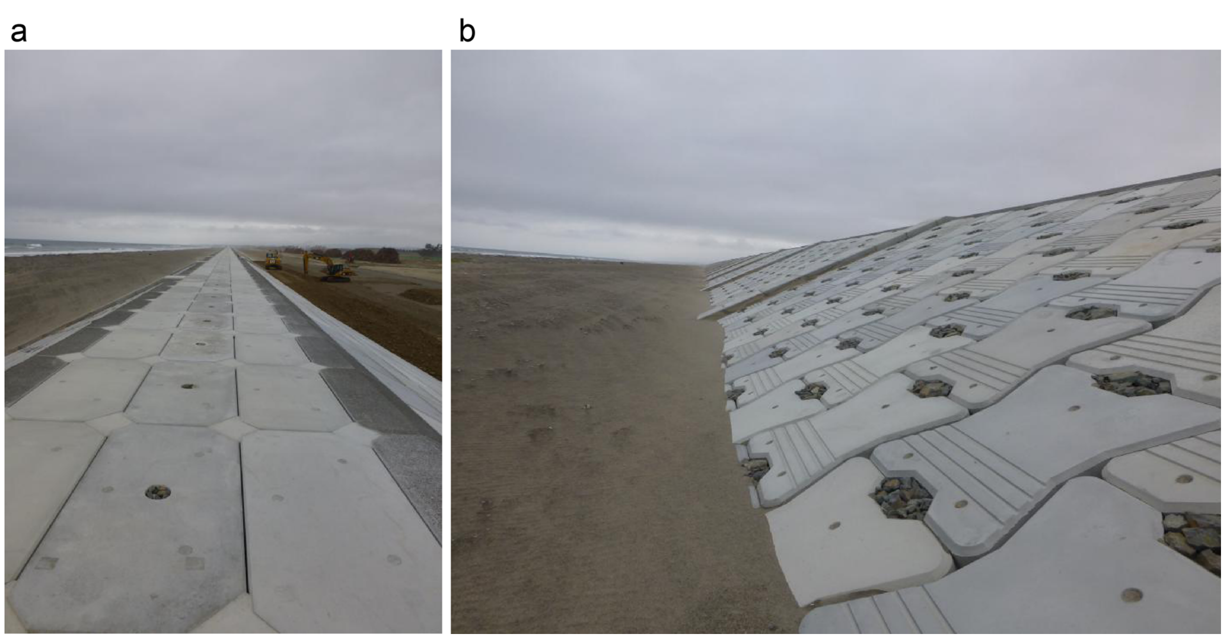

Fig. 13. Recently completed sea dikes defending Sendai airport just south of Yuriage village, Natori City: (a) looking along the crest of the structure southwards; and (b) details of the interlocking armour units on the offshore side of the structure.

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Fig. 14. Arahama sea dikes: (a) new portion under construction; (b) offshore side showing extent of construction site; (c) end of newly-completed portion south of the construction site; (d) looking southwards along completed portion.

no interaction with other countries though there are similarities in the documents. Further international initiatives will have taken place since this, especially around the Eurocodes (the set of harmonised technical rules being implemented in Europe), though waves attributable to seismic movement are not yet considered in the Eurocodes [8].

The World Association for Waterborne Transport Infrastructure or PIANC are a body of international experts on technical, economic and environmental matters. Their Working Group 53 studies recommendations of maritime construction in areas prone to tsunamis, with a view to disseminating information to port designers and operators. Following the 2011 tsunami, PIANC produced a report based on the work that Japan's Port and Airport Research Institute conducted which included detailed site surveys and observations [54]. The closing section of the report discusses new worst-case scenarios of tsunami sources and new approaches to design of coastal defensive structures and coastal towns.

In the US, the design and construction of sea defence structures is undertaken by the US Army Corps of Engineers who use the internationally reputed Coastal Engineering Manual (CEM) [5] as their technical guidance document. The manual is written by a panel of experts under the auspices of the Coastal and Hydraulics Laboratory. The CEM [5] pre-dates both the 2004 Boxing Day tsunami and the 2011 Japanese event which may explain the lack of reference to tsunamis, but it is still notable by its absence for a country at significant risk from this threat [66]. The manual does refer to a 1980 publication by Frederick Camfield of the US Army Corps of Engineers Coastal Engineering Research Centre [6] which gives quite detailed design principles. Some of these principles strike a chord with recent tsunami findings: the protective effect of vegetation (citing research dating back to 1918); the position of breakwater to avoid unwanted reflections; damage mechanisms including scour; and the danger of receding water. However, whilst there is good detail for onshore forces – for debris flow and onshore structures – there is nothing on forces on coastal defensive structures. The listed damage mechanisms focus mainly on erosion and the only wave pressures presented are equivalent to hydrostatic values *i.e.* there is no allowance for dynamic or impact effects, which is not surprising given the understanding of impulsive pressure at that time.

The US Federal Emergency Management Agency (FEMA) have produced guidelines for the design of vertical evacuation structures, *i.e.* buildings towards which a population would evacuate in areas of low-lying ground and where the tsunami source is in close proximity. The most recent revision, FEMA P-646 [20], includes lessons learned from the 2011 tsunami; these are not discussed here as they do not pertain to sea defences. The CEM and other guidelines have no force of law; for legal requirements it is necessary to consider national codes and their referenced American Society of Civil Engineers (ASCE) standards. There are more than 60 ASCE standards covering all aspects of design, construction and maintenance of civil engineering projects. Of relevance here is the ASCE 7 *Minimum Design Loads for Buildings and Other Structures*, produced by the ASCE, which is currently being revised. A sub-

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Fig. 15. Main damage chain of gravity-based structures due to tsunami (translated from NILIM disaster scenario manual [43]).

committee entitled *Tsunami Loads and Effects* is producing details of tsunami loads on a range of structures for a new chapter. The material is informed in part by a visit by the ASCE Structural Engineering Institute team that travelled to Japan after the 2011 tsunami to assess the performance of a range of structures [9]; the subcommittee was coincidentally authorized about a month before the 2011 tsunami. This standard is due to be issued late 2015/early 2016 and will be mandatory for states of Alaska, Washington, Oregon, California and Hawaii. Several design maps for these regions will be produced, including the probabilistic offshore tsunami amplitudes based on the concept of a Maximum Considered Tsunami, corresponding to a 2500 return period event. This design basis event is used to provide inundation depths and flow velocities for Tsunami Design Zone maps. It is understood that this code will not explicitly contain information on the design of sea defence structures though the proposed Section 6.7 does contain comprehensive guidance on the prediction of tsunami amplitude, run-up and inundation, including such factors as coastal subsidence. It also addresses detailed numerical modelling issues such as model resolution and essential physics to be included *e.g.* reflected waves, channelling in bays and bore formation. Predictions based on this guidance could form the basis of the design of coastal defence structures in a tsunami-prone region.

5.2. UK design experience

In the UK there are different levels of regulation for designers of sea defences. Firstly there are Acts of Parliament, supported in some areas by Statutory Instruments. Providing more specific, though not mandatory design guidelines, are the British Standards (BS), some of which are codes of practise whilst others give specifications [22]. At a UK PIANC conference in 2014 [55] the revised suite of BS6349 (Maritime works) were suggested to be close to completion and perhaps forming the basis of a possible Eurocode

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Fig. 16. Pair of photographs taken in the EEFIT missions across the port of Minamisanriku: (a) 2011 and (b) 2013.

or ISO standard. At the moment there is no Eurocode that specifically covers Marine or Coastal Structures, though there are Eurocodes that have general applicability to coastal structures *e.g.* EN 1991: (Eurocode 1) Actions on structures and EN 1997: (Eurocode 7) Geotechnical design. A significant challenge of updating BS6349 has been to ensure consistency with relevant European harmonised standards [63]. Within BS 6349-1-1:2013 tsunamis are mentioned in an appendix on metocean data acquisition and are described as an example of infrequent condition, with a warning that a short to medium-term data set might miss this type of event.

Aside from these codes there are a number of guidance documents at the disposal of British coastal engineers, the most recent of which are: The European Overtopping Manual [57] (currently being revised), The Rock Manual [10], The Toe Structures Management Manual [4] and The Use of Concrete in Maritime Engineering: A Good Practise Guide [11]. Tsunami do not feature in these publications; even the Toe Structures Management Manual [4] which was published subsequent to the 2011 tsunami only mentions them in connection with research undertaken by a Japanese researcher [47] that related scour depth to overtopping. That said, they do address the most important failure mechanism identified following the Japanese 2011 tsunami *i.e.* toe scour. Other European countries have similar situations in terms of using a blend of European standards and local and international guidance.

In addition to design guidelines many European countries have conducted hazard assessments. For example, as a result of the 2004 Boxing Day tsunami the UK government commissioned reports to assess whether the country was at risk from tsunami [13,14]. The first report had identified four potential source origins and had produced initial model predictions of wave heights around the coast. The subsequent report focused on two of those source origins: the North Sea and a 1755 Lisbon-type event and investigated the south-west and western Ireland regions in more detail with estimations of wave heights and celerities. The worst-affected areas (west Cornwall) might see an increased sea level of 4 m but a value of 2 m is more typical, both from the Lisbon-type event. The lower water levels are comparable to storm surges though the wave form and hence impact on the shoreline will be different [48].

5.3. *Design of sea defence structures for nuclear power facilities*

Nuclear installations (power and non-power) had been subject to Periodic Safety Reviews and have taken into consideration the UK DEFRA tsunami studies [50]. Following the 2011 Japanese tsunami and particularly the effects of the nuclear disaster at Fukushima Daiichi, the HM Chief Inspector of Nuclear Installations implemented reviews of all nuclear installations, reporting to the Secretary of State for Energy and Climate Change. This move was closely followed in the European Council by the requirement for "stress tests" of nuclear power stations to be undertaken; these were to involve the investigation of "extreme natural events challenging plant safety functions". This was implemented by the European Nuclear Safety Regulators Group. The process was extended in the UK to cover all licensed nuclear facilities [50]. Arising from the HM Chief Inspector of Nuclear Installations interim report are a series of recommendations that include Interim Recommendation 10 (IR-10) that says:

> "*The UK nuclear industry should initiate a review of flooding studies, including from tsunamis, in light of the Japanese experience, to confirm the design basis and margins for flooding at UK nuclear sites, and whether there is a need to improve further site-specific flood risk assessments as part of the periodic safety review programme, and for any new reactors. This should include sea-level protection*." [49]

In the Final Report from the HM Chief Inspector of Nuclear Installations [48] it is recognised that since the DEFRA studies [13,14] there has been further research undertaken into tsunami sources, such as potential submarine landslides further north in the Arctic, into tsunami propagation (particularly in connection with sea level rise) and into historic tsunami which may all lead to revisions in water levels due to tsunami in the UK.

Nuclear power continues to be a major contributor to the UK's energy mix with Europe's largest construction project, the £24.5bn Hinkley Point C now under construction [21]. Built immediately to the west of the Hinkley Point A and B stations in the Bristol Channel, it will have a 900 m long seawall [15] of crest height 13.50 m Above Ordnance Datum (AOD) with an integrated footpath set at 12.40 m AOD as indicated in Fig. 17. The Hinkley Point C Stress Test report as described by the ONR [50] (covered by the non-power stress tests as it was at that time a *future* licensed site) gives a flood height diagram, showing the extreme flooding level of 9.52 m AOD (with no waves) and the site platform at 14 m AOD. Taking into consideration "wave effects" of 2m this gives a margin of 2.68 m for the site though a little less than 2 m before overtopping of the wall would occur. This is deemed to be sufficient as the site is a small distance inland of the seawall. Électricité de France (eDF), who run the Hinkley power stations, commented that natural hazards including all combinations of tides, storm

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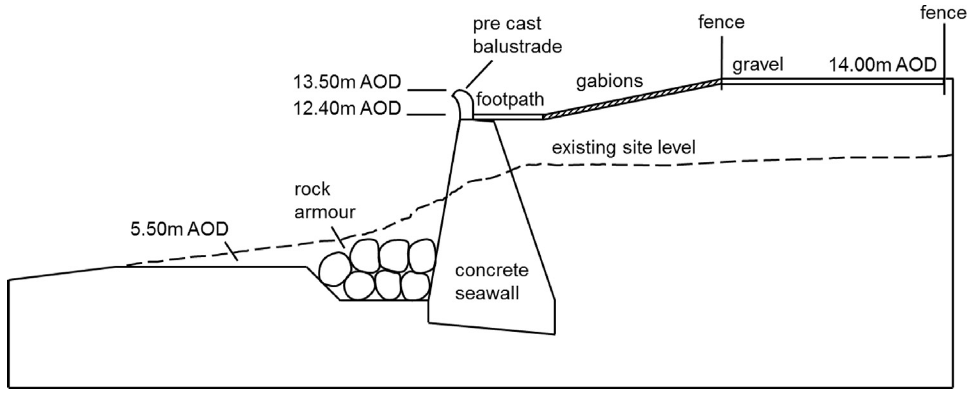

Fig. 17. Section through the proposed seawall at Hinkley C power station.

surges, tsunami and possible sea level rise due to climate change have been taken into consideration [16]. eDF also commented that the hazard return period is 1 in 10,000 years and a design life of 60 years has been used. In fact, the seawall at Hinkley Point C is by no means dwarfed by the newly constructed 22 m wall that defends the Hamoaka nuclear power station visited by EEFIT in 2013 [18], despite the much greater tsunami inundation levels.

The Hamoaka nuclear power station is situated about 200 km south-west of Tokyo. Fig. 18(a) is a photograph of the model of the newly completed Hamoaka seawall that protects the facility (taken in the Exhibition Centre at the power station since photography was not permitted outside this area). It shows the various stages of defence from the concrete armour units on the beach to the vegetated dike and finally the seawall, constructed from reinforced concrete, steel cladding and a 70 mm layer of render (at prototype scale). Fig. 18(b) shows how the design of the wall evolved from a +18m T.P. crest to a +22 m T.P. crest with an additional section and reinforcement; this crest level is now assumed sufficient to prevent tsunami overtopping resulting from a Nankai Trough Megaquake, the source of which is in close proximity to the power station. However, in case the tsunami is of such a size that it overtops the wall there are additional measures to prevent reactor building flooding (*e.g.* additional walls, watertight doors) and to ensure continued cooling function (*e.g.* back-up gas generator on higher ground, additional water tanks). The Onagawa nuclear power station which lies closer to the epicentre of the 2011 earthquake than the Fukushima Daiichi plant has seawall crest levels of 17 m though there are now plans for a super-seawall with a crest level of 29 m, designed to withstand a 23 m tsunami. The wall is being built immediately landward of the existing wall and comprises a 'steel pipe-type' construction of length 680 m. It is due for completion by March 2016. These are examples of seawalls that have been designed to withstand Level 2 events, for obvious reasons.

In the report Foresight Reducing Risks of Future Disasters: Priorities for Decision Makers [25] written for the UK Government Office for Science the authors do not identify tsunami as contributing to coastal flooding, listing only storms as the source mechanisms. They also state that the UK approach is different from that of the Japanese, being based on goal-setting "rather than a purely deterministic, prescriptive, methodology". According to Day and Fearnley [12] permanent mitigation measures that include seawalls of a certain height are critically dependant on an accurate determination of the extreme water levels. There is also the danger that getting this estimation wrong inhibits responsive mitigation *e.g.* warning systems, citing examples of people who had evacuated to prescribed 'safe' locations following the 2011 but perishing due to inaccurate estimations. Had they used their instincts and gone elsewhere they may have survived. Day and Fearnley [12] call this a "brittle" mitigation strategy.

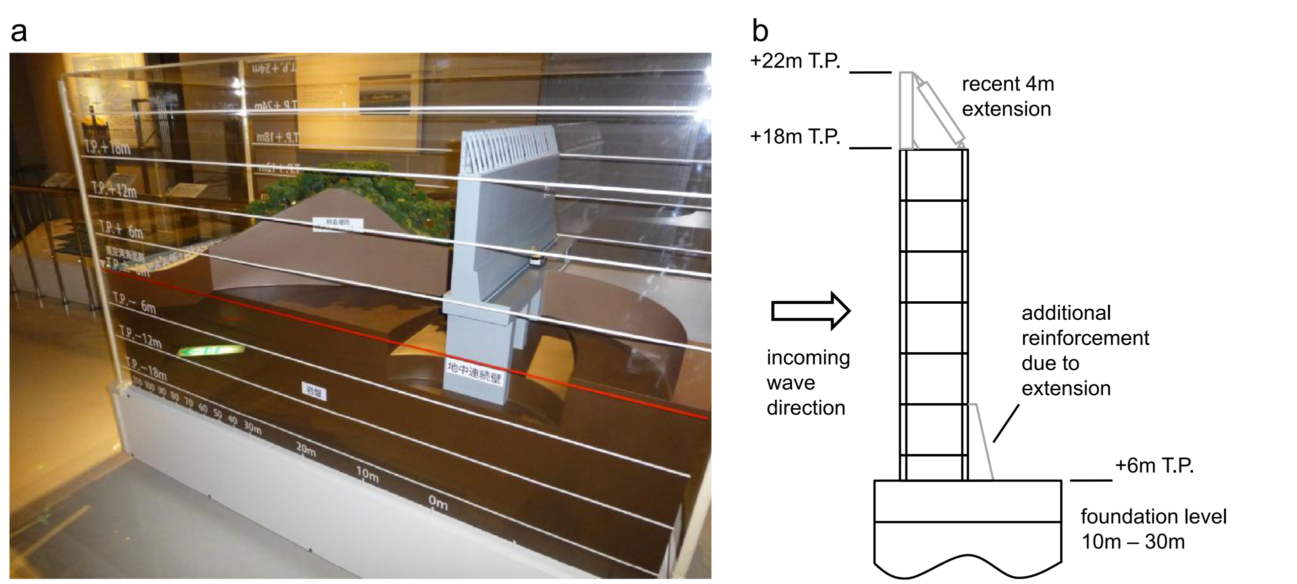

Fig. 18. Hamaoka nuclear power station seawall: (a) Exhibition Centre model of the final design and a (b) section through wall with recent modifications illustrated in grey.

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6. Conclusions

Japan's rich history of sea defence design has been enshrined in standards and supplementary manuals. However the unexpected size of the 2011 Great East Japan earthquake and tsunami led to the failure of a great number of sea defence structures including: the brand new tsunami defence breakwater at the industrial port of Kamaishi; the sea dikes defending the international airport at Sendai; the 10 m seawall in Tarō (renowned for the performance of its seawall in 1960); and most critically the seawall that protected the Fukushima Daiichi nuclear power station.

Lessons learned from these failures have been publicised in research articles (both nationally and internationally), in new Japanese engineering guidelines and manuals, and in revisions to their standards. Work has been undertaken by NILIM (that supports MLIT), the Port and Airport Research Institute, international bodies such as PIANC, researchers at many universities and by reconnaissance mission teams from both inside and outside Japan. There is evidence of cooperation between many of these organisations *e.g.* the ASCE reconnaissance team was hosted by the Japanese Society of Civil Engineers; Japanese academics have been involved in developing the manuals and in publicising new approaches in international journals and at conferences; and in the fact that one individual in NILIM is responsible for the revision of standards and manuals for Japan. All this should lead to a consistent body of material useful for the designer.

A key change in the design approach is the adoption of a two-level description of tsunami hazard based upon return periods, with sea defence structures being required to protect (*e.g.* prevent overtopping) from the lower level hazard. However the structure must merely remain intact in the event of a higher level as this should still offer a degree of protection. Non-structural measures would complement the sea defences for both levels of hazard.

One seminal document, the NILIM disaster scenario manual [43], is comprehensive in its description of the effects of tsunamis on a range of sea defence structures. This material enables the designer to appreciate the possible failure mechanisms. New recommendations in the standards and manuals include: the use of geotextile membranes in sea dikes to avoid leaching of infill material; reinforcement of sea dike toes on the landward side; widening of breakwater rubble mounds; and interlinkage/reinforcement of concrete blocks in seawalls.

The EEFIT return mission in 2013 provided opportunities to visit newly-completed sea defences. Considerable progress on replacement sea dikes at two locations along the coastal plains was observed, though seawalls at Minamisanriku remained untouched as the population had now moved inland. There was evidently progress on the repair of the Kamaishi breakwater and contacts at PARI indicated there was agreement to rebuild the Tarō seawall.

Material available to designers of sea defence structures in other tsunami-prone areas has been considered, though there appears to be a paucity of such information. This is understandable in some regions where less developed countries face competing pressures for limited financial resources, but it is notable that this threat is not addressed in design codes for at-risk European countries (despite a risk and precedent of large tsunamis in the region). In this region there needs to be more joined-up thinking between those who understand the tsunami sources and the implications for populations and infrastructure. In the US there will shortly be the revised ASCE 7 standard, a significant resource for designers, but this material stops short of explicit guidance for the design of tsunami-resistant sea defences.

Shock waves that reverberated around the nuclear industry following the Fukushima Daiichi nuclear catastrophe are forcing designers to consider these risks but outside of Japan, design guidance seems to be lacking. It is essential that the findings resulting from the tragic events of March 2011 are disseminated as widely as possible, both to inform industrialised nations and those that rely on international codes.

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