FAILURE MECHANISMS AND LOCAL SCOUR AT COASTAL STRUCTURES INDUCED BY TSUNAMI

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Abstract

On March 11 2011, a substantial tsunami was triggered by a massive earthquake that occurred offshore, in the northeast coast of Japan, which affected coastal infrastructure such as seawalls, coastal dikes and breakwaters in Tohoku region. Such infrastructure was built to protect against the level 1 tsunamis that previously hit the region, but not for events as large as the 2011 tsunami, which was categorised under level 2 tsunamis (Shibayama et al. 2013). The failure mechanisms of concrete-armoured dikes, breakwaters and seawalls due to level 2 tsunamis are still not fully understood by researchers. This paper investigates the failures modes and mechanisms of damaged coastal structures in Miyagi and Fukushima prefectures, following the authors' post-disaster field surveys carried out between 2011 and 2013 after the Tohoku tsunami. Six significant failure mechanisms were distinguished in sea dikes and sea walls: 1) Leeward toe scour failure, 2) Crown armour failure, 3) Leeward slope armour failure, 4) Seaward toe and armour failure, 5) Overturning failure and 6) Parapet wall failure, with leeward toe scour being recognised as the major failure mechanism in the majority of surveyed locations. Furthermore, it was revealed that many of the breakwater failures were likely caused by the differences in hydrostatic pressure between the front and back of the breakwaters. It was also found that breakwaters in Ishinomaki failed due to uneven settlement at the base caused by the back face toe scour or liquefaction seismic action. The authors proposed a simple practical theoretical model for predicting the scour depth at the leeward toe, by integrating the model with surveyed field data and also the hydrodynamics of tsunamis. Moreover, a Large Eddy Simulation (LES) model was employed to further support the key findings from the field surveys and predictive model results. Finally, key recommendations are given for the design of resilient coastal defence structures that can survive Level 2 tsunamis in future.

Key words: 2011 Tohoku tsunami; Coastal structures; Post-disaster field surveys; Scour failure; Failure modes and mechanisms; theoretical model; LES model.

1. Introduction

On 11th March 2011, at 2.46 pm in Japanese Standard Time, North East Japan (Tohoku region) was shaken by a moment magnitude (Mw) 9.0 subduction earthquake, which triggered a massive tsunami wave. This was one of the most severe tsunamis recorded in the history of Japan, and it was categorised as a Level 2 tsunami (Shibayama et al. 2013), with a return period of once every thousand years. The earthquake ruptured in an area nearly 300×500 km2 and its epicenter was located approximately 130km east of Sendai City, Miyagi Prefecture, and 32km below the ocean's surface. According to the information published by the Japanese Meteorological Agency (JMA), dislocation of the land upward was 1 m vertically and 5 m horizontally. According to the preceding tsunami records, it was seen that the incident wave heights and the run-up heights observed in this event were exceptionally high, with wave and run-up heights rarely ever reaching such values. The 2011 Tohoku Earthquake Tsunami Joint Survey Group observed maximum tsunami inundation and run-up heights varying between 20 to 40 m. The reported death toll was about 15,800 people, with 3962 missing and US$ 300 billion worth of assets being lost by this catastrophic event (EERI Special Earthquake Report, 2011).

Many post-tsunami field surveys carried out in the affected regions showed that the coastal defense structures along the Japanese coast line were not strong enough (in terms of structural stability and hydraulic performance) to safeguard the infrastructure and the low lying coastal communities at most locations. As these natural phenomena are extremely unpredictable, it is wise to strengthen the mitigation measures to resist even a level 2 tsunami event, as they may occur in the future. In order to implement precise measures in resilient structures it is important to carry out an in-depth investigation of the failure modes and the mechanisms in defence structures.

According to the New York Times [2011], more or less 43% of Japanese coastline was defended from tsunamis, abnormal wind waves or storm surges by concrete seawalls, earth filled/rubble armoured sea dikes or a combination of both types. According to post-tsunami field investigations, laboratory experiments and the numerical modelling carried out by the coastal researchers in the field, on the 2011 Tohoku tsunami and other tsunamis, it is evident that the factors which affect to the failure modes and mechanisms of the coastal defences are beach topography, coastal geomorphology, structure geometry, wave, current and sediment characteristics. Through detailed post-tsunami surveys, many research teams around the world examined the structural stability of coastal defense systems against the wave loads. Methods of high resolution LIDR (LIght Detection and Ranging) were employed by some of the researchers (e.g. Yim et al. 2012) to collect acute geospatial data to integrate with observations, topography and measurements in coastal structural failure to design and validate models under tsunami and structural loadings. Thus, results are being used to design more resilient structures by accurately quantifying the damage.

Furthermore, significant attempts have been made in recent studies to define and verify the complex phenomena, and also to mitigate the catastrophic damage induced by massive events such as the 2011 Tohoku Tsunami. The outcome of these numerous studies are implemented in building codes, helping to upgrade the standards of safety and strengthening structures to withstand tsunami induced forces and currents. According to the model experiments carried out by Mizutani and Imamura [2002], it was found that significant impact forces are induced by the overflowing wave pressures on the stability of the coastal defences. Such extensive results led to the development of various types of numerical simulations, which help to understand the failure phenomenon and in turn, aid in implementing engineering measures to alleviate the damage caused by tsunamis. Moreover, many studies highlighted that scour around the coastal defence structures was the dominant failure mode by which coastal defence schemes failed catastrophically in 2011 Tohoku tsunami. Researchers such as, Van et al. [2007], Mazumder [2007], Zhao et al. [2010], Yeh [2010], Kato et al. [2006, 2012], Yeganeh-Bakhtiary et al. [2012], Arikawa et al. [2012] and Aryan and Shingan [2012] were involved with experimental studies which covered the scour mechanism around the vertical cylinders and breakwaters. There is however a need of a greater understanding with regard to the scour failure mechanism in various types of coastal defences in order to initiate thorough mitigation measures.

Moreover, Kato et al. [2006] described the major failure processes of the sea defences caused by the scour around the leeside and seaside of the defence walls, and also due to the exerted wave forces on the parapet walls during the tsunami run-up and draw-down processes. The hydraulic laboratory experiments carried out by Kato et al. [2006] revealed the relationship between over flowing depth, the extent of the scour and the forces involved in the scouring of leeward slope and the toe. Furthermore, Kato et al. [2012] carried out a post-tsunami field survey for the 2011 Tohoku tsunami across the coastal vicinity of Amori and Chiba Prefectures and confirmed the leeward toe scour was the leading failure mechanism in defence structures in that area. Further, eight overall failure mechanisms were revealed in their extensive study and they estimated that 49.2% of the defence failure caused by the scouring at the leeward toe.

According to the detailed field surveys carried out by Jayaratne et al. [2013] in Miyagi and Fukushima Prefectures, it was highlighted that scour at the leeward toe was the most prevalent failure mechanism due to 2011 Tohoku tsunami. Shuto et al. [2009] summarised that the tsunami defence walls failed due to the tsunami wave overflow and the backwash based on the detailed observations carried out over the post-tsunami records of Great Sanriku [1933] and Chilean tsunami [1960]. Mori et al. [2011] conducted a post-tsunami field survey in the Tohoku region after the massive event and verified that coastal geomorphology had a considerable effect on the failure modes of coastal defences.

Scour around the sea defences is caused partly by the tsunami wave run-up and the draw-down processes. The exact effects that each process has however cannot be verified by a post-tsunami field survey. Also it is not possible to be entirely precise on aforementioned process that drives the rapid scour in coastal defences. In a scale model, Tonkin et al. [2003] observed that for a sand substratum, major scouring occurred during the draw-down process where the flow was highly turbulent. At the end of the draw-down process, pressure on the subsoil was decreased and pore pressure gradient was uplifted. This caused a reduction in the effective stresses between soil particles where the shear stress induced by further wave action could rapidly increase the scour depth. This analysis was further verified by Yeh et al. [2004] in their study.

Moreover, extensive studies carried out by Yeh [2010] on the tsunami scour mechanism revealed that the momentary liquefaction in subsoil increases the amount of scour that occurs during the tsunami wave draw-down process, and due to reduced shear strength of soil substrate, it becomes like a heavy liquid that is more susceptible to scouring. FEMA [2000, cited by Francis, 2006] proposed a graphical relationship between inundation flow velocity and the depth of scour despite the consideration on tsunami flow and the soil characteristics. Further, Moronkeji [2007] described the presence of excessive pore pressure gradient where the void ratio was increased, hence the initiation of soil scour took place.

The Reynolds-Averaged Navier-Stokes (RANS) equation, which stimulates steady flow, was utilised to investigate the scour process around a submerged cylinder by Zhao et al. [2010]. The RANS equation was coupled with a bed morphological model where transport rates of suspended load and the bed load were considered. Further, Zhao et al. [2010] observed a negative exponential relationship between the change in rate of scour depth and height of the cylinder.

Thus, it is evident from the numerous previous studies that observation and analysis of the prevailing, and other hydraulic, failure mechanisms in coastal defences is a significant task as it aids in implementing mitigation measures and in establishing a safer environment for coastal communities in the hinterland area. The main focus of this paper is to elaborate on the investigated (main and other) failure mechanisms and the governing physics. Also, the various parameters involved in each of the failure modes will be analysed, and finally the outcome of the numerical modelling will be produced and a simple theoretical model for predicting scour depth at the leeward toe will be derived, with the help of the post-disaster field surveys of Jayaratne et al. [2013]. This theoretical model is predominantly influenced by the tsunami hydrodynamics, soil properties and the physical geometry of the structure**.** Further, the authors believe that this study will be useful in the development of stable coastal defence systems that will be able to alleviate the damage that may be induced by future tsunami events.

2. Post-Tsunami Field Surveys by Jayaratne et al. [2013]

As a part of research collaboration between University of East London, UK and Waseda University, Japan, Jayaratne et al. [2013] conducted a detailed post-disaster field reconnaissance in the period between 2011 and 2013, to understand the variety of potential failure mechanisms which were involved in the catastrophic failure of coastal defence schemes in North Eastern Japan due to the 2011 Tohoku tsunami. The investigations were mainly focused on collapsed coastal defences, including breakwaters in the Miyagi and Fukushima prefectures of the Tohoku region where massive devastation occurred due to the 2011 event. The authors gathered detailed measurements of scour features and geometry of the damaged structures, sketches, digital photos and videos, geographical information (e.g. longitudes and latitudes) and the tsunami deposit soil samples at the broken defence structures. The other published literature was also used to collect information regarding the hydrodynamics of tsunami data for present analysis.

By the detailed investigation performed upon collected field data, it was clearly noticeable that the defence schemes at the examined locations had undergone several failure mechanisms. This was determined by studying the observed tsunami wave heights, geometry and strength of the defences and the beach topography where structures were located. The Ministry of Land, Infrastructure, Transport and Tourism [MLIT] of Japan stated in their post-tsunami investigation that the coastal defences which catastrophically failed by the Tohoku tsunami were only designed to resist characteristics of tsunamis with return periods of up to 120 years; tsunamis categorised under Level 1 (Shibayama et al. 2013) and the typhoon-induced storm surges. Even though distinctive characteristics such as coastal morphodynamics which relate to the oceanic topography, wave force interaction and the coastal processes have been involved in the design aspects of these defences, they failed catastrophically under the abnormal forces generated by this massive event.

Further, during this post-tsunami reconnaissance, the authors observed a variety of wave and inundation heights and massive impact forces exerted by the tsunami waves on defences at the surveyed locations. Being closer to Sendai bay as well as to the earthquake epicenter, coastal defences at Miyagi prefecture experienced substantial damage due to the large tsunami waves. It is also believed that seismic motion had significant effects on the stability of structures and also contributed to the damage of the defence systems. It could be seen that the Fukushima prefecture had also suffered relatively severe damage, including failures to defences and the Daiichi nuclear power station.

Figure 1 is a location map showing the sites, throughout the Miyagi and Fukushima Prefectures, at which coastal defences were examined for probable failure mechanisms caused by the 2011 Great East Japan Earthquake and Tsunami.

(INSERT Figure1)

Table 1 shows the surveyed locations of post-tsunami field investigations carried out by Jayaratne et al. [2013] in Miyagi and Fukushima prefectures, which were severely damaged due to 2011 Tohoku tsunami. Section 3.1 covers the rigorous analysis of the failure mechanisms of coastal defence structures and tsunami wave characteristics, wave forces that acted on the defence structures, stability and strength of the structures, tsunami deposits and the geomorphology of the surveyed locations necessary to draw the final conclusions.

(INSERT Table 1)

3.1 Failure Modes and Mechanisms of Seawalls and Sea Dikes

Analysis of failure modes and mechanisms were carried out by the observations made through field measurements and digital photos taken at the aforementioned post-tsunami field surveys. In the Miyagi and Fukushima prefectures, six notable failure modes and mechanisms were identified in the coastal defence structures. The authors recognised leeward toe scour as the failure mechanism that was most likely responsible for the total failures of structures in the majority of surveyed locations. Also, it was revealed that most of the failures were not only governed by the tsunami-induced forces but also many other forces acted together to cause the total collapse of the defence schemes.

The major tsunami induced failure modes by which the observed defence structures failed are as follows:

1) Leeward toe scour failure

2) Crown armour failure

3) Leeward slope armour failure

4) Seaward toe and armour failure

5) Overturning failure

6) Parapet wall failure

3.1.1 Leeward toe scour failure

This failure mode was recognised as the most prevailing failure mode that was found in the coastal defences during 2011 Tohoku tsunami. Leeward toe scour is governed by the wave run-up and draw-down processes where massive wave pressure is generated on the structure. Laboratory experiments carried out by Mizutani and Imamura [2001] showed that there were four types of pressure fields that were induced by the tsunami wave run-up process. These were: dynamic, sustain, impact standing and overflowing wave pressures. The authors also stated that the overflowing wave pressure may act as the main cause of leeward toe scour failure. The physical process behind the leeward toe scour failure could be explained as follows:

- After applying the overflowing wave pressure on the toe, wave draw-down process takes place.
- At the last stage of the draw-down process, pore pressure gradient in the toe bed is promptly increased as the velocity and water level decrease, where the low pressure field is induced.

- Effective stress between soil particles becomes considerably low (Tonkin et al. 2003).
- Consequently, due to higher surface shear stress generated by the rapid tsunami flow over the slacken soil easily erodes the toe of the structure.
- Subsequent cyclical tsunami inflow and outflow may create significant scour around the coastal structures. If the leeward toe scour continues towards the dike's body, the leeward slope armour will lose its structural support which will result in it being removed completely from the structure or broken due to successive wave currents and wave action. This could lead to total collapse of the defence structures.

In field survey performed by Jayaratne et.al [2013], along the coastal line of Miyagi and Fukushima prefectures, two types of coastal defence systems were observed: Earth filled armoured dikes and re-curved concrete seawalls. Two possible failure modes were observed in the two types of defences.

(INSERT Figure 2a)

(INSERT Figure 2b)

(INSERT Figure 2c)

(INSERT Figure 2d)

Figure 2(a) depicts the failure of a sea dike in Iwanuma city. This failure was initiated by the leeward toe scour which was caused by the massive tsunami overflow. Successive wave action created a large scour area and eventually, the leeward armour was damaged due to the loss of backing. Wave pressure and concentrated wave currents undermined the dike body and some parts of the dike were completely washed away due to hydrodynamic forces that exerted via the open ends. Figure 2(b) shows a severely scoured dike toe and the damaged body at Soma city which was caused by tsunami wave action. In Figure 2(c), a partially damaged sea dike at

Shichigahama is shown. The damage has been inflicted by the leeside toe scour, but the successive waves were not enough to completely destroy the dike body in this location. Figure 2(d) shows the schematic sketch of the physical mechanism involved in leeward toe scour failure.

As mentioned previously, in this post-tsunami field survey, it was seen that leeside toe scour led to quite different types of structural failure in re-curved seawalls than in the earth filled coastal dikes. Figure 3(a) and 3(b) clearly illustrate the different failure processes caused by the leeward toe scour in re-curved seawalls, in Watari (North) and Soma city respectively. Once scouring occurs and undermines the foundation of the seawall, the structure becomes destabilized, making it more susceptible to failure by overturning and/or sliding when successively impacted wave run-up and drawdown processes. Figure 3(c) depicts the removal of the pavement at the leeward toe due to scouring at Ishinomaki (East). Figure 3(d) shows the expected stages of leeward toe scour failure in re-curved seawalls.

(INSERT Figure 3a)

(INSERT Figure 3b)

(INSERT Figure 3c)

(INSERT Figure 3d)

3.1.2 Crown armour failure

This failure mode was seen to be the second most dominant mode in damaged sea dikes during the post-tsunami field surveys. Due to tsunami waves, hydrodynamic forces and wave currents, the crown armour was uplifted and scattered elsewhere. As successive waves seeped in and scoured the exposed crown, the dike body eventually collapsed totally. In order to understand the physical phenomena of this failure mode in greater depth, Kato et al. [2012] carried out laboratory experiments and observed a suction force which was created by the negative pressure on the crown edge during the wave overflow. It could be assumed that due to this negative pressure, crown armour may be stripped off from the crest and failure could be caused. This can also be rectified as bore type tsunami wave fronts always comprise of irrotational flow along with positive vorticity, which generates suction force. Crown armour may also experience the wave characteristics seen in bore type tsunami wave fronts, which maintain until wave breaking takes place. When the induced suction force was greater than the stabilizing forces keeping the armour units in place, crown armour failure occurred. The exposed dike crest was subsequently eroded by further wave inflow and outflow and as a result, total collapse often occurred.

Figures 4(a) and 4(b) depict the top and side views of a collapsed sea dike at Yamamoto city respectively. It is evident from Figure 4(a) that the highest pressure concentration occurred at the leeside edge of the crown; hence the tarmac layer was stripped off. The sequential failure pattern was created at this stage due to the existence of the concrete beams that were evenly placed underneath. Figure 4(b) illustrates the severely eroded dike body which was damaged by subsequent rapid wave seepage through the exposed dike crest. Moreover, one could predict that the dike fill might have been pulled out from the exposed crown due to the continual suction force that acted on the dike crest. It could also be assumed that once the dike sections were damaged, hydrodynamic forces imposed by the succeeding wave action would have led to a rapid collapse of the entire dike.

Figure 4(c) shows the exposed crown due to removal of crown armour layer of the sea dike at Soma city and also shows the progressive scour of the dike filling across the opened crown. Fig. 4(d) depicts the initiated similar failure pattern in a dike at Ishinomaki (West).

(INSERT Figure 4a)

(INSERT Figure 4b)

(INSERT Figure 4c)

(INSERT Figure 4d)

3.1.3 Leeward slope armour failure

The third significant failure mode observed in this post-tsunami field survey was leeward slope armour failure. As mentioned before, according to the experiments performed by Kato et al. [2012], it was reported that the highest magnitude of negative pressure acted on the crown edge of the leeward slope. Therefore, it is possible that this failure mode was caused by negative pressure, which would have resulted in the leeward slope armour being stripped off from the edge of the crown and then scattered. One could predict that once the leeward slope armour is removed due successive wave pressure, constant impact from tsunami currents and negative forces could have led to the total collapse of the structure through constant erosion of the exposed leeside slope. Mizutani and Imamura [2002] found, through methods of hydraulic laboratory experiments, that in order to investigate the dynamic tsunami wave forces acting on the structure, overflowing wave pressure on leeward slope of a sea dike is essential. Hence, it is reasonable to assume that the leeward slope armour failure is also regulated by the overflowing wave pressure by developing rapid erosion. When this failure process alone is considered, there are no signs of toe scour at the leeward slope.

Figures 5(a) and 5(b) show the captured images from two cities in the Miyagi prefecture which highlight traces of leeward slope armour failure due to 2011 Tohoku tsunami. Damaged leeward slope of both dikes were covered by thick polythene layer and there was no observed leeward toe scour in both locations. Figure 5(c) shows the steps of leeward slope amour failure in a rubble mound structure in the event of a tsunami.

(INSERT Figure 5a)

(INSERT Figure 5b)

(INSERT Figure 5c)

3.1.4 Seaward toe scour and armour failure

During the post tsunami field survey, seaward toe scour and armour failure was not so commonly observed in the surveyed locations, however there were a few instances where structures were assumed to have failed by this particular failure mode. It is predicted that seaward toe scour is caused in the same way as seaward toe scour, but induced by particularly at the tsunami draw-down process. Due to continuous seaward toe scour under constant wave inflow and outflow conditions, seaward armour may lose its backing given by the toe. Subsequently, the armour would be detached and scattered, leaving the slope exposed to further erosion from waves and eventually leading to total collapse Also, it is possible to suggest that tsunami driven deep wave troughs could act on the seaward toe and induce scouring. It could also be predicted that seaward armour failure may occur as a result of wave water seepaging through the armour and scouring the inner mound. This would destabilize the structure, scattering its armour and resulting in total collapse.

Figures 6(a)-6(c) show traces of this failure mode at Higashimatsushima, Watari (North) and Kitakami respectively. It was noticed that revetments were under construction at Higashimatsushima and Watari (North) whereas destabilised seaward slope armour were seen in coastal defences at Shichigahama. Figure 6(d) illustrates the possible stages of this type of failure mode.

(INSERT Figure 6a)

(INSERT Figure 6b)

(INSERT Figure 6c)

(INSERT Figure 6d)

3.1.5 Overturning failure

This failure mode can be observed particularly in concrete seawalls rather than in sea dikes, in fact the shape of the wall and materials are significant factors affecting this mechanism. From the information collected throughout the surveyed locations, it was found that overturning failure seldom led to the total collapse of coastal defence structures. However, a few instances were found in the Miyagi and Fukushima Prefectures. Overturning failure occurs when there is a large difference in hydrostatic pressure between seaside and leeside water levels. This induces lateral thrust, which destabilises the structure. Also, due to the wave forces induced by the wave run-up, draw-down and wave slamming, a destabilising moment is exerted at the toe of the defence. When the resisting moment stirred by the dead weight of the wall is exceeded by the destabilising moment, seawall tends to be overturned (see Figure 7a). The seawall could be overturned either towards the leeside or the seaside based on the resultant moment. If the wall is overturned by the wave run-up, it will fall on to the leeside while if it is caused by wave draw-down process, the wall will fall on to the sea side (Figure 7b). Moreover, soil liquefaction under foundations of seawalls due to seismic activities and toe scour in lee and seaside of walls are generally followed by overturning failure due to further tsunami wave action.

(INSERT Figure 7a)

(INSERT Figure 7b)

3.1.6 Parapet wall failure

A parapet wall is an additional curved wall extension which is generally connected to the crown of the sea dike or standing alone from the shoreline in order to reduce the volume of water entering to the hinterland by wave run-up and overtopping. During the post-tsunami field surveys, coastal defences with parapet walls were rarely observed, i.e. only at Soma city, but where they were seen, parapet wall failure was commonly found. This failure mode did not directly lead to the total collapse of whole defence systems, but only reduced the strength of the structures. Parapet wall failure is mainly forced by the wave run-up and draw-down processes (Kato et al. 2006). The process by which parapet walls fail can be outline as follows: when the shear force exerted by the tsunami wave exceeds the shear strength of the parapet wall section, the wall fails. If the parapet wall is damaged as a result of the wave run-up process, the broken parapet wall section would fall on the leeside and if the failure is caused by the wave draw-down process, the wall section would be found on the sea side. This failure mode is also often caused by the slamming/impact forces of incoming waves and also the horizontal pressure applied on the parapet wall by impulsive waves with shorter periods. Wave born debris and direct wave impact loads are also responsible for causing cracks on the parapet walls where the compressive strength of the wall is exceeded by the wave compressive force and subsequently breaching takes place by further wave action. Figure 8(a) depicts the partially broken parapet wall at Soma city while Fig. 8(b) illustrates two distinctive types of parapet wall failures.

(INSERT Figure 8a)

(INSERT Figure 8b)

3.2 Failure Modes and Mechanisms of Armoured Breakwaters

An essential role is played by the offshore breakwaters located along the Japanese coastline as they help dissipate the disastrous forces generated by tsunami waves. In the event of 2011 Tohoku tsunami, offshore breakwaters suffered significant damage and some parts completely collapsed. Many factors influence the damage of breakwaters, however hydrostatic pressure difference between leeward and seaward faces acts as the main failure mechanism (Arikawa et al. 2012). Hydrostatic forces are created when the tsunami hits because there is a difference in water level either side, and this is what created the lateral thrust by which scatter primary armour away. Subsequently, hydrodynamic pressures and currents induced by tsunami waves caused sections of breakwaters to be removed and scattered far off from the original location of the breakwaters. Rapid increase in pore pressure gradient between adjacent armour units due to massive water seepage is also a reason for displacing primary armour units. Other physical processes of failing of breakwaters, particularly in caissons, are by sliding or overturning due to the hydrostatic pressure difference and the impacting force of successive wave action. Further, bed scour driven by the overflowing wave pressure and the higher flow velocities at the base of the back face and sides of the breakwaters also tends to lead the structure to fail.

During post-tsunami field surveys by the authors, significantly damaged breakwaters were found at the Ishinomaki commercial port. Seven massive breakwaters were observed which were made of concrete armour units, extending east to west side, 150 m away from the sea dike on the shore. Considerable damage was noticed in the breakwaters located on the east side (four breakwaters) which were almost sunk. From this it is possible to assume that due to the back face toe scour or seismic action, bed soil liquefaction occurred, which resulted in uneven settlement at the base of the structure. In the west side, the remaining three breakwaters have missed armour units probably induced by the lessened effective stresses due to enhanced pore pressure gradient and wave currents.

Figures 9(a)-9(d) show photos of a series of offshore breakwaters at Ishinomaki commercial port which were captured during the field surveys, and through before and after tsunami images from Google images. These figures reflect the magnitude of damage that was inflicted on the breakwaters in this area.

(INSERT Figure 9a)

(INSERT Figure 9b)

(INSERT Figure 9c)

(INSERT Figure 9d)

4. Mathematical Modelling

4.1 Modelling Procedure

It was evident from the field investigations of Jayaratne et al. [2013] that the most probable mode by which structures failed in the Miyagi and Fukushima prefectures was due to scour at leeward toe. The authors have attempted to derive a simple predictive model for scour depth of coastal structures with the help of measured scour depth, sediment and geometrical properties of structures and the published information of tsunami parameters. The mathematical modelling procedure used to develop the model is described below.

(INSERT Figure 10)

The mathematical modelling technique was used to explain the scour damage caused by tsunami based on the Buckingham π theorem and the model experiment performed by Mizutani and Imamura [2002] to calculate the impact overflowing wave pressure behind a sea dike.

The dependent variables of the scour phenomenon are:

$$D_s = f(D_s, \rho, g, H_{d2}, P_{om}) \qquad (1)$$

where D_s is the measured scour depth, ρ is the sea water density, g is the acceleration due to gravity, H_{d2} is the height of structure measured on the leeward side, and P_{om} is the maximum overflowing wave pressure. From dimensional analysis, the non-dimensional scour depth $\frac{D_s}{H_{d2}}$ is found to depend on the inverse of maximum overflowing pressure, $\frac{\rho g H_{d2}}{Pom}$, an equation already derived by Mizutani and Imamura [2002]. The mathematical form of the event is shown as:

$$\frac{D_s}{H_{d2}} = f\left(\frac{\rho g H_{d2}}{Pom}\right) \qquad (2)$$

The equations for maximum wave pressure behind a sea dike derived by Mizutani and Imamura [2002] from a model experiment of bore overflowing a dike on level bed:

$$\frac{P_{om}}{\rho g H_{d2}} = 2\sqrt{2}\,\frac{V_m \sin\theta_2}{\sqrt{gH_{d2}}} \qquad (3)$$

where V_m is the maximum velocity on the dike crest; taken as inundation flow velocity behind the structures, and θ_2 is angle of landward slope.

For case where $2\sqrt{2}\,\frac{V_m \sin\theta_2}{\sqrt{gH_{d2}}} > 1$;

Dynamic overflowing wave pressure can be derived from,

$$\frac{P_{om}}{\rho g H_{d2}} = (2\sqrt{2}\,\frac{V_m \sin\theta_2}{\sqrt{gH_{d2}}})^4 \qquad (4)$$

The need to estimate tsunami velocity during run-up process is based on the need to determine the fluid force on structures and other tsunami flow parameters. Inundation depth, bottom slope, bottom roughness, water surface slope and distance from shoreline are factors that make it difficult to derive an estimate velocity value (Matsutomi et al. 2010). Following the 2011 Tohoku tsunami, Chock et al. [2013] employed captured video and satellite imagery tools to analyse flow velocity, but due to the unavailability of videos at the exact locations under consideration, the maximum flow velocity over the structure, V_m, given in Eq. [3] is assumed to be the upper bound inundation flow velocity derived by Matsutomi et al. [2010]. This velocity exerts the largest fluid force and is related to the inundation height (h), as $U=1.2\sqrt{gh}$. This changes Eq. [4] to:

$$\frac{P_{om}}{\rho g H H_{d2}} = (3.4\frac{\sqrt{h}\sin\theta_2}{\sqrt{H_{d2}}})4 \qquad (5)$$

In order to quantify the performance of the predictive model, Root Mean Square (RMS) error and Scatter Index (SI) were used. Mathematically they are expressed as $\sqrt{\frac{\sum_{i=1}^{N}(Yi - Xi)^2}{N}}$ and $\frac{RMSE}{\bar{X}}$ respectively, where Y_i is the model parameters, X_i is the observed parameters, $\bar{X}$ is the mean value of the observed data and N is the number of measurements.

Dimensions taken off the sea dikes and seawalls were used in the wave pressure calculations at all locations. The scour depth measurements recorded were the representative maximum values along the stretch of the coastal structures surveyed. The Coefficient of Permeability (k) of the soil grains at leeward side of coastal structures were derived using particle size analysis in order to set the boundary condition for the predictive model and thus preserve its applicability range.

(INSERT Table 2)

4.2 Sediment Particle Analysis

The Particle Size Distribution (PSD) curve derived from sieve analysis results was plotted using the UK convention and presented in Table 3. The curve reveals that the soil grains are predominantly single-sized sand because the particle size ranges from 0.063–2.0 mm. The results of Uniformity Coefficient (U) and coefficient of curvature (Z) which relate to the general shape and slope of the PSD curve and describe the smoothness and shape of the gradation curve respectively, are presented in Table 2. Since the samples are uniformly graded, the Darcy's Coefficient of Permeability (k) which is a measure of the ease with which water can flow through the voids of soil particles, could depend on the void size, which is related to particle size. This empirical relationship for clean filter sand, is given by Hazen (1892: cited by Powrie, 2004) as:

$$k\ (m/s) = 0.01\ D_{10}^2 \qquad (6)$$

(INSERT Figure 11)

(INSERT Table 3)

These values of Coefficient of Permeability denote a high void ratio which implies the possibility of pore pressure increase during tsunami inundation, which in turn would increase rapid scour (Yeh et al. 2004). It can be deduced from the figure above that Ishinomaki (East) has the highest coefficient of permeability and surely the lowest void ratio or porosity of the five locations. Since the grains are uniform graded sand with high void ratio, it is safe to conclude that there would be not much difference in void ratio property for samples at Soma, Watari, Yamamoto and Iwanuma since they are of the same order of magnitude.

4.3 Representative Scour Depth Predictive Model

It focuses on the development of a new scour depth predictive model using the maximum overflowing wave pressure model of Mizutani and Imamura [2001] to develop appropriate geometry that will improve the structural effectiveness of sea dikes. This model was combined with other non-dimensional terms using Buckingham pi theorem and scour depth field data to derive this model. It can be observed from the trend of the data that a decreasing exponential function seems to exist between the relative scour and the inverse of relative impact overflowing pressure. The average error of the initial function produced by MS Excel 2007 spreadsheet was too large and because of this the model function was adjusted by using its Solver tool.

From Fig. 10, the relative mean scour depth becomes:

$$\left.\frac{D_s}{H_{d2}} = 0.85\left(\exp\, -\left(16\,\frac{\rho g H_{d2}}{P_{om}}\right)\right)\quad\right\}\qquad \frac{Pom}{\rho g H_{d2}} > 0 \qquad (7)$$

Inserting Eq. [4] into Eq. [5], the final model that incorporates easily measurable in-situ quantities is generated and given below.

The scour depth prediction equation can be re-written as:

$$\left.\frac{D_S}{H_{d2}} = \lambda\left(\exp - \left(\frac{\sqrt{H_{d2}}}{2\lambda\sqrt{h}\,\sin\theta_2}\right)^4\right)\quad\right\}\qquad \begin{array}{c} h > 0 \\ 10^{-4} < K < 10^{-3} \end{array} \qquad (8)$$

where a fitted coefficient (λ) is found to be 0.85.

The result shows there is an exponential relationship between relative measured scour depth, height of structures measured on leeward face, Inundation height/flow velocity and angle of leeward slope. The function is defined for boundary conditions when the inundation height is greater than 0, and coefficient of permeability (k) is greater than 10^{-4} but less than 10^{-3}. It is crucial to note that this model is influenced by the presence of offshore breakwaters. Also, high relative scour suggests high extent of structural damage at the landward face and toe. This was noticed at Ishinomaki (East), Soma city and Watari town. Figure 12 is based on the assumption that by applying Buckingham π theorem, relative impact overflowing wave pressure is related to the relative scour depth at those locations. It suggests that high impact overflowing wave pressure will generate a high relative scour depth, with the highest relative scour depth at Ishinomaki (East).

(INSERT Figure 12)

The following deductions can be made from the predictive model:

- For any given angle of landward slope, higher sea dike will minimise soil scour.
- The higher the inundation height, which is a function of flow velocity, the higher the possible scour depth. This concurs with the general trend that high flow velocity will cause high shear stress which in turn causes scour.
- High sea dike and seawall with mild slope tends to decrease scour depth. This agrees with sea dike design approach of a landward face with gradients 1:2 (26.6^o) and 1:3 (18.4^o) of Netherlands (Linham and Nicholls, 2010).

The RMSE and SI values of 0.24 and 0.38 respectively suggest that the model is reasonably well in predicting a representative scour depth around a concrete sea dikes and seawall.

5. Numerical Modelling

A one-phase (liquid-phase) Large Eddy Simulation model of Mikami and Shibayama [2013] was applied at laboratory-scale to highlight the large wave overflowing pressures and inundation flow velocities generated at the leeward slope and toe of the structure. The governing equations are the spatial filtered Navier-Stokes equations along with the continuity equation as shown in Eqs. [9]-[10]. The total flow velocity, u_i (i=1, 2, 3) consists of two components; spatially averaged velocity ($\bar{u}_i$) and small-scale turbulence velocity ($u_i^{/}$).

$$\frac{\partial \overline{u_i}}{\partial t} + \bar{u}_j \frac{\partial \overline{u_i}}{\partial x_j} = -\frac{1}{\rho}\frac{\partial \bar{p}}{\partial x_i} + \nu \frac{\partial^2 \overline{u_i}}{\partial x_j x_j} - \frac{\partial \tau_{ij}}{\partial x_j} + g_i \qquad (9)$$

$$\frac{\partial \overline{u_i}}{\partial x_i} = 0 \qquad (10)$$

where $\bar{u}_i$ and $\bar{u}_j$ are the i^{th} and j^{th} velocity components spatially averaged over the grid size respectively, t is the time, x_i (i=1, 2, 3) is the co-ordinate spacing, τ_{ij} is the Sub-Grid Scale (SGS) stress, $\bar{p}$ is the pressure, g_i is the gravitational acceleration, ρ is the density of fluid and ν is the molecular viscosity.

The Sub-Grid Scale (SGS) stress (τ_{ij}) can be written as the product of i^{th} and j^{th} small-scale turbulence velocities (Eq. [11]).

$$\tau_{ij} = \rho\overline{u_i^{/} u_j^{/}} = \rho(\overline{u_i u_j} - \bar{u}_i\bar{u}_j) \qquad (11)$$

After examining several SGS models, the Smagorinsky Model (Smagorinsky, 1963) was selected as it is widely used by many LES models.

$$\tau_{ij} = -2\nu_e \bar{D}_{ij} \qquad (12)$$

where ν_e is the viscosity coefficient for the SGS model and $\bar{D}_{ij}$ is the strain rate.

The viscosity coefficient (ν_e) and the strain rate ($\overline{D}_{ij}$) are given by Eqs. [13] and [14] respectively.

$$\nu_e = (C_s\Delta)^2\sqrt{2\overline{D}_{ij}\overline{D}_{ij}} \tag{13}$$

$$\overline{D}_{ij} = \frac{1}{2}\left(\frac{\partial \bar{u}_i}{\partial x_j} + \frac{\partial \bar{u}_j}{\partial x_i}\right) \tag{14}$$

where C_s is the Smagorinsky constant, ranging from 0.07-0.21 and Δ is the spatial length scale given in Eq. [15].

$$\Delta = (\Delta x_1 \Delta x_2 \Delta x_3)^{1/3} \tag{15}$$

where Δx_i is the grid spacing in the i^{th} direction.

The Cubic Interpolated Pseudo-particle (CIP) method, initially described by Yabe et al. [1990] is employed to solve the governing equations and the Successive Over-Relaxation method is employed to solve the pressure equation. The free surface elevation is calculated using the density function method following the work of Watanabe and Saeki [1999]. The density function (*f*) is defined based on '1' and '0' assigned for 'grid cell with water' and 'grid cell without water' respectively and calculated using CIP method with 1.0×1.0 cm grid size. Equation [16] shows the density function used in the numerical computations.

$$\frac{df}{dt} = 0 \tag{16}$$

The model verification was done with the experimental data of the collapse of a water column gathered by Martin and Moyce [1952] and shown in Mikami and Shibayama [2013]. It can be concluded that a good agreement was found between experimental results and predicted values from the numerical model.

A laboratory-scaled (1:50) coastal dike model with a wave wall (circled area), generally found in coastal defences of Miyagi and Fukushima prefectures was initially simulated to explore the regimes of high overflowing pressure and velocity areas as shown in Fig. 13(a). Figure 13(b) depicts geometrical configuration and initial condition of a dike with presence of artificial armour (circled area), without a wave wall.

Figure 14(a)-(d) illustrates the time variation (t=1.0-4.0 s) of overflowing pressures as the tsunami wave propagates over the seaward and leeward slopes. When t=2.0 s significant pressure builds up on the seaward slope reaching a maximum value of 3000 N/m^2 at the leeward toe as indicated in Fig. 14(b). When t=3.0 s tsunami flow decelerates and run-down process takes place leading to less pressures such as just over 1000 N/m^2 being generated at the leeward toe [Fig. 14(c)]. This value further reduces to about 250 N/m^2 when t=4.0 s [Fig. 14(d)].

(INSERT Figure 13a)

(INSERT Figure 13b)

(INSERT Figure 14a)

(INSERT Figure 14b)

(INSERT Figure 14c)

(INSERT Figure 14d)

The numerical simulations of overflowing pressures with presence of artificial armour show a different scenario than the case with presence of wave wall. When t=2. 0 s the pressures acting on artificial armour and seaward face of coastal dikes has reached about 2000 N/m^2 however the pressures at leeward toe has reduced to about 1500 N/m^2 [Fig. 15(b)]. Interestingly, this value is further reduced to about 1000 N/m^2 when t=3.0 s [Fig. 15(c)]. Finally, when t=4.0 s, the overflowing pressures have gone down as lower as 250 N/m^2 [Fig. 15(d)].

(INSERT Figure 15a)

(INSERT Figure 15b)

(INSERT Figure 15c)

(INSERT Figure 15d)

(INSERT Figure 16a)

(INSERT Figure 16b)

(INSERT Figure 16c)

(INSERT Figure 16d)

Figure 16(a) illustrates tsunami flow velocities such as 1.5 m/s approaching the front face of the coastal dike with a wave wall. When t=2.0 s similar magnitudes of velocities (around 1.5 m/s) are generated over the leeward slope and at the toe [Fig. 16(b)]. These velocities are further reduced between 1.0-1.5 m/s when t=3.0 s as seen in Fig. 16(c). During draw-down process of tsunami (t=4.0 s), velocities are reduced to about 0.5 m/s at the leeward toe [Fig. 16(d)]. The zones of 'no velocities' just after the wave wall are shown by the numerical model and it is considered to be a limitation of the present model.

(INSERT Figure 17a)

(INSERT Figure 17b)

(INSERT Figure 17c)

(INSERT Figure 17d)

Figure 17(a) illustrates high velocities (~1.5 m/s) of tsunami wave approaching artificial armour when t=1.0 s. When t=2.0 s similar magnitudes of velocities induce in upper layers of water but leave with lower velocities such as about 0.5 m/s on the crest, leeward slope and at the toe [Fig. 17(b)]. When t=3.0-4.0 s the velocity values in the upper water layer of crest and leeward slope become about 0.75 m/s and it is kept at 0.5 m/s at the leeward toe [Figs. 17(c)-(d)].

6. Lessons Leant for Resilient Structures in Future Tsunamis

The implementation of mitigation measures is a largely challenging issue in tsunami susceptible areas around the world as the extent and strength of tsunamis are unpredictable and almost unprecedented. The 2011 Tohoku tsunami triggered the utmost damage and had extreme wave heights which were found to be the highest recorded since the record keeping started in Japanese tsunami history.

Thus, requirement of dependable resilient structures which carry sustainable structural attributes has become vital in order to resist intensified forces driven by the tsunami waves. In terms of wave forces, hydrodynamic forces, hydrostatic, impulsive, buoyant and debris impact forces are potentially the most problematic in a devastating event such as the Tohoku tsunami. These forces are, in most occasions, acting collectively on the structure to make the total collapse. Hence, it is necessary for these resilient structures to be resistant against all of these forces.

With detailed analysis carried out upon gathered data in post-disaster surveys by the authors, it was defined the highest possible failure mechanism was leeward toe scour. According to the surveys carried out by Jayaratne et al. [2013] in the Tohoku region, many mitigation measures have already been implemented to form a sustainable and resilient structural scheme along the Japanese coastline, particularly to protect the leeward toe as well as other failure mechanisms in future events.

6.1 Mitigation measures for leeside slope and toe failure

According to the observations made in 2013, it is possible to observe that additional fortifying measures have been implemented in order to protect the defence walls from leeward toe scour. Figure 18 presents the leeward toe construction pattern in damaged areas where defence systems collapsed critically due to leeside toe scour. Thick and heavy concrete frames were laid and filled with concrete to form a protective toe at the leeside and concrete panels were placed on the frames to cover the toe with sealed edges. These protecting measures help the structure to resist against the scouring that occurs at the leeward toe. The sufficiently sealed toe will counteract the water seepage into the subsoil. Also, heavy concrete frames withstand the up-thrust given by generated pore pressure gradient during tsunami draw down in case water seepage is occurred.

Many potential measures have been utilised in order to protect the leeward slope from the suction forces and the subsequent scouring of the dike body. . The leeward slope was covered by the heavy concrete armour units in order to resist the uplifting negative forces, which are induced by bore type tsunami waves, and also in order to resist against the scattering of the armour by the wave currents. The crown edge, an area where the negative pressures acted strongly, was sealed sufficiently to provide sound resistance. Concrete plates were well-sealed in order to slow down/reduce the substantial wave water seepage to the dike body. The weep holes however were kept to provide room for wave energy dissipation via percolation of a little wave water through the aggregate fill. Figure 18(a)-18(c) depict counter measures taken for leeward toe scour and slope failure of newly constructed dikes at Yamamoto and Iwanuma cities as well as dikes under-construction at Watari (North).

(INSERT Figure 18a)

(INSERT Figure 18b)

(INSERT Figure 18c)

6.2 Mitigation measures for crown armour failure

Major counter acting hydraulic force on stability of the structure in this failure mode is negative pressure exerted on the crown. Wave water seepage through the exposed dike crown, increased pore pressure gradient and the reduced effective stress also govern the failure. Hence, crown should be sufficiently protected to resist these factors.

During the last survey, it could be noticed in newly built sea dikes, crown was well protected by heavy concrete plates and they were properly sealed around the edges. Therefore, these measures will resist the suction forces and reduce the wave water seepage which increases the pore pressure gradient. The weep holes on the crown pave the room for wave water to seep in and dissipate energy through the dike body.

Figure 19(a)-(c) illustrates the newly built crown of coastal dikes in Yamamoto and Iwanuma cities and a cross sectional view of the proposed crown at Watari (North) respectively.

(INSERT Figure 19a)

(INSERT Figure 19b)

(INSERT Figure 19c)

6.3 Mitigation measures for seaward slope armour and toe failure

Similar to the above mitigation measures, there is a new protection measure for this type of failure mode. Figure 20(a)-(c) shows the employed reinforcing as counter measures. It was noticed in the proposed design drawings and in the sea dikes that are under construction, that the seaward toe is being further protected by coarse aggregates and a concrete cover. This was in order to sustain the wave forces and to restrict the subsequent increase of pore pressure gradient by wave water seeping into the sub soil. Furthermore, in order to withstand the wave induced forces and currents which detach and scatter the armour units, the aggregate layer was strengthened by using heavy concrete armour units.

(INSERT Figure 20a)

(INSERT Figure 20b)

(INSERT Figure 20c)

7. Concluding Remarks

On March 11, 2011 at 2.46 pm (JST) Japan Northeast coast, known as Tohoku region was devastated by gigantic tsunami driven by a subduction earthquake which was marked in Richter scale as 9.0M. This event was categorised as Level 2 tsunami (Shibayama et al. 2013) and the biggest natural disaster ever experienced by the Japanese coastal community. The authors conducted post-disaster field surveys in the period of 2011-2013 in Miyagi and Fukushima prefectures which were enormously battered by this event, with the aim of detailed investigation of failure modes and their mechanisms, leading to development of predictive model for scour depth of coastal structures.

Following detailed analysis upon collection of field data, six major failure modes were defined within surveyed coastal defences in Miyagi and Fukushima prefectures. Those are as follows:

1. Leeward toe scour failure
2. Crown armour failure
3. Leeward slope armour failure
4. Seaward toe scour and armour failure
5. Overturning failure
6. Parapet wall failure

It was identified that leeward toe scour was the major failure mode which led to the catastrophic failure of coastal defence systems in the Miyagi and Fukushima prefectures. Due to negative pressure exerted by the bore type tsunami waves on the crown of the sea dikes and leeward slopes, armour failure occurred and subsequent wave action led to the total collapse of structures. The other failure modes were less commonly observed in the surveyed vicinities.

It was also observed that mitigation measures were implemented in the Tohoku region in order to strengthen the sea defences and highest possible failure which is leeward toe scour is significantly concerned by the Japanese coastal engineers. In order to mitigate the other failure mechanisms, counter measures were taken to reinforce the entire coastal defence system. Heavy concrete panel segments were installed as provisional mitigation measures in fortifying the coastal defence systems against wave induced forces and currents.

During the field surveys, breakwater failure mechanisms such as overturning, sliding, up-thrust driven by the enhanced pore pressure gradient, scour at the leeward foundation rip-rap and scattering of armour due to drag force induced by wave currents were also identified and investigated. Among the surveyed locations, breakwaters at Ishinomaki port were found to have failed by the scattering of armour units and sinking of breakwaters. It was expected that this failure may have taken place as a result of uneven settlement of foundation rip-rap due to scour driven by rapid flow velocities at the leeward toe and its edges.

Apart from the detailed field surveys that were carried out in order to define the failure modes and physical mechanisms, mathematical modelling was also performed to understand the soil-hydrodynamic-structure interaction. It is evident that the scour depths and extent depend on the characteristics of tsunami wave, soil and structure geometry. A simple theoretical model was developed using governing physics of tsunamis, and the Buckingham π theorem as a reliable practical tool which predicts scour depth around leeward toe of a concrete coastal defence structure.

At this level, the Coefficient of Permeability (k) and sand grain diameter (D_{10}) are considered to be the representative parameters for initiation of sediment motion (e.g. Shields parameter) which is directly related to soil scour. Further research is needed in order to understand this issue.

Finally, the one-phase Large Eddy Simulation (LES) model, developed by Mikami and Shibayama [2013], was employed at laboratory-scale examples to support the findings from field surveys, particularly to reflect large pressures and velocities built up at the leeward slope and toe area.

The key conclusions made up on theoretical model outputs are outlined below:

- In order to define the best suited design, model results can be obtained for the range of physical geometry of sea defences. The best tolerable model could finally be employed to develop as an ideal or standard structure for the sea dikes and concrete seawalls and it is possible to propose for the tsunami prone regions.

- Factors which strongly govern scour depth are, permeability of soil, landward slope angle, dike and seawall height which is measured from leeside toe and inundation height which is a function of inundation flow velocity.
- Highest relative mean depth of scour is generated by the highest dynamic overflowing wave pressure.
- According to the model output, integration of small seaward slope and taller sea dikes/seawalls are the best suited geometry to mitigate excessive scour at an event like the 2011 Tohoku tsunami.
- Leeside toe and slope are essential parts of a coastal structure that are to be heavily protected against the massive volume of tsunami overflow.

8. Future Work

Classification of complex scenarios of scour failure modes and mechanisms in different geometries of coastal structures under tsunami attack is essential to be explored through highly controlled 2D and 3D laboratory experiments. This will be conducted in the Hydraulic Laboratories at the University of East London, UK and at Waseda University, Japan. The one-phase Large Eddy Simulation (LES) model developed by Mikami and Shibayama [2013], will be further modified to simulate field-scale numerical predictions of scour depths at leeward toe of structures.

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