EVALUATION OF LATERAL AND BUOYANT FORCES ON REINFORCED CONCRETE BUILDINGS BY THE TSUNAMI OF THE 2011 EAST JAPAN EARTHQUAKE

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SUMMARY:
Tsunami waves associated with The Great East Japan Earthquake on March 11, 2011, caused serious structural damages to reinforced concrete building structures, such as story collapse and base overturning. In this paper, the equivalent loads on the building structures were estimated from the damages in relation with the velocity and buoyancy of the tsunami. The drag force on building structures can be given by a function of the tsunami velocity, which was correlated with a distinction between inundation depths in front and behind of the building structures. The overturning moment resistances at the bases were calculated from the gravity weight and the tensile resistance of piles. Considering the buoyant force, the calculated resistance is compared with the overturning moment by the lateral tsunami loads. The calculated results were verified for the four-story overturned reinforced concrete building with the pile foundation.

Keywords: Tsunami Damage survey Drag force Buoyant force Overturning

1. INTRODUCTIONS

Great Tsunami was observed in a widespread coastal area in Tohoku by Great East Japan Earthquake. The measured inundation depth of tsunami exceeded 10 m in several areas, and not only wooden houses but also reinforced concrete buildings or steel buildings collapsed or overturned by tsunami load in those areas. In May 2011, Building Research Institute has been carried out joint research in a committee on the structural design of tsunami evacuation buildings (Leader: Prof. Y. Nakano, the University of Tokyo) under Japanese building code revision promotion project, which mainly focuses on appropriate design tsunami loads for tsunami evacuation building.

In this study, tsunami load to damaged and non-damaged building structures is evaluated from a ratio of water depth, which gives equivalent hydrostatic force to the structural strength, to the measured inundation depth (water depth ratio). Other parameters were not taken into account because (1) few building structures have structural damage when buildings are taller than tsunami, (2) damaged buildings concentrated on particular regions, and that makes difficult to quantify a relation between water depth ratio and over flood water amount. However, the number of survived buildings tends to increase in proportion to the measured inundation depth in spite of identical water depth ratio in above statistical study.

This paper reports on the detail analytical studies on two reinforced concrete building structures, which has been affected by tsunami loads basically assumed in the design of tsunami evacuation building structures. Overturning of the four story building structures with pile foundation are also analysed considering buoyant force and tensile strength of piles.

2. LATERAL LOAD OF TSUNAMI

The tsunami loads on buildings is supposed to be hydrostatic pressure of three times inundation depth in “Japanese design guideline for tsunami evacuation building 2005 (Fig. 1.)”, which was verified from experimental study or tsunami damage survey in 2004 Sumatra Earthquake. However, this amplification factor “3” deserves the maximum value for the inundation depth, which takes into account for an effect of high tsunami velocity and breaking wave. The actual tsunami loads on building structures basically shows smaller value than this estimates, and depends both on inundation depth and tsunami velocity. The acting tsunami force on objects in fluid (i.e. concrete pier) is generally given by a drag force in Eqn. 2.2. The momentum of tsunami is generally given by a Froude number Fr, which is a ratio of flow velocity (v) to the square root of product of inundation depth and gravity acceleration ($(gh_w)^{-0.5}$). Two Equations means the hydrostatic force (a=1 in Eqn. 2.1) is equivalent to the drag force when Froude number is 1.4.

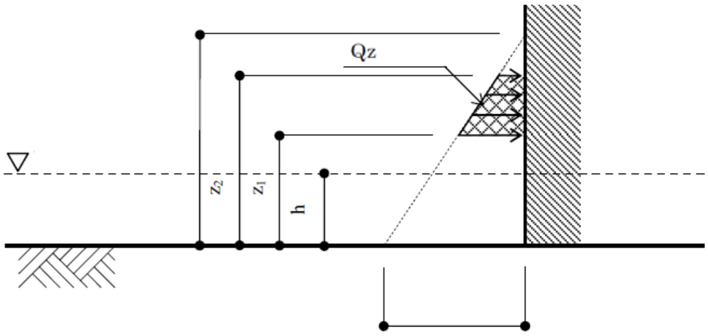


Figure 1. Japanese design guideline for tsunami evacuation building 2005 (a=3.0)

$$P_S = \int_0^{h0} (\rho g(ah_w - z)B(1-\zeta))\,dz \qquad (kN) \tag{2.1}$$

here, a: water depth ratio, ρ: density of water, g: gravity acceleration, h_w: inundation depth, B: building width, h0: the minimum value within inundation depth and building height, ζ: aerial opening ratio

$$P_F = B\int_0^{h0} (0.5C_d\,\rho v^2 B)dz \qquad (kN) \tag{2.2}$$

here, v: flow velocity, C_d: drag coefficient (= basically 2.0)

$$v = C_v\sqrt{2g(h_f - h_b)} \qquad (m/s) \tag{2.3}$$

here, h_f: inundation depth in front of an object, h_f: inundation depth behind an object, C_v: flow velocity coefficient (= basically 1.0),

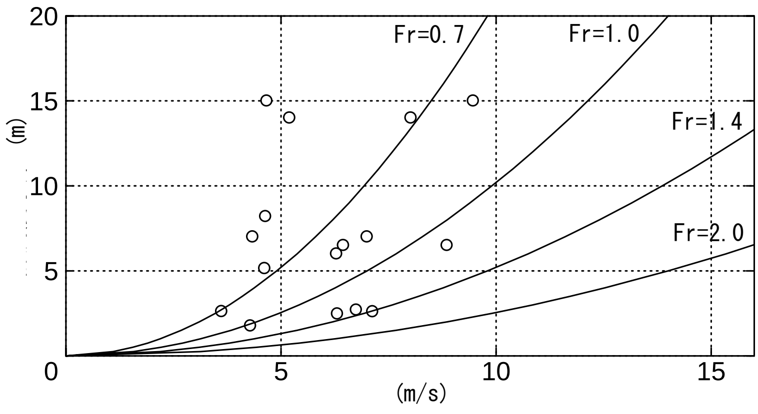


Figure 2. Comparison between inundation depth and flow velocity in several regions

Fig. 2. shows a relation between flow velocity and inundation depth behind buildings in several regions. Flow velocity was estimated with theoretical formula proposed by Matsutomi (Eqn. 2.3), which is derived from distinction of inundation depths in front and behind buildings. For inundation

depths data, it refers to a tsunami database provided from a joint research group on 2011 Tohoku Great Earthquake in this study. As shown in this figure, Froude number seems to be around 0.7 on this Tohoku Great Earthquake, when measured inundation depth is larger than 10 m. The hydrostatic force of inundation depth ($0.7 \times 1.4 \fallingdotseq 1.0$) is adopted for tsunami loads in following analyses.

3. ESTIMATE OF TSUNAMI LOAD FROM DAMAGED BUILDING

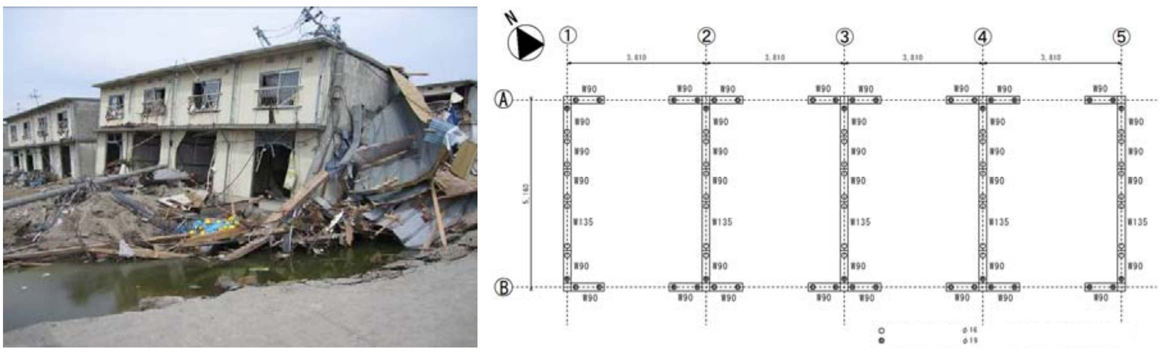
3.1. Residential building A

3.1.1. Outline of the building

The residential building A is two-story precast concrete panel structures built in 1970 (Fig. 3 (a)), and located in Yuriage district of Miyagi prefecture. There are nine similar residential buildings around this building, which consists in three to six units. The 1st floor plan is shown in Fig. 3 (b). The building has four spans in longitudinal (X) direction, and one span in transverse (Y) direction. The span length is 3.81 m in X direction, and 5.16 m in Y direction. The total height of this building is 5.85 m, which was smaller than measured inundation depth in this area. The diameter of anchorage bolts for concrete panels is 16 or 19 mm. There was no opening in transverse elevation. The aerial ratio of opening to the longitudinal elevation shows larger value for windows and entrance. Tsunami ran up from the South-East coast 800 m far from this buildings, and lateral loads was mainly induced in transverse direction. There was sufficient vacant space in front and aside of the building and an effect of surrounding buildings are negligible.

3.1.2. Comparison between Strength and Inundation depth

The building frame inclined with erosion of surrounding soil affected by tsunami, but there was no structural damage both in longitudinal and transverse direction. Among surrounding similar buildings, a part of section collapsed for the erosion of soil, or concrete panels broke by debris impact, but a tsunami wave pressure did not cause obvious damage to those structural frame as well. The residual tsunami traces was observed at 7.5 m height from ground level in front elevation of surrounding three story building structures. The water trace was also observed at 5.0 m height in that building. Building Research Institute estimates the tsunami flow velocity in this district to 7.0 m/s based on the television pictures of Tsunami.



(a) overview of building A (b) Plan of residential building A

Figure 3. Remained residential building A

The structural weight and lateral load carrying capacity of the building was calculated based on the Japanese design guideline for prefabrication residential houses with precast concrete panels. The weight per unit area ω can be derived from Eqn. 3.1. The floor area is 78.6 m^2 both in 1st and 2nd floor, and estimates of total weight is 971 (kN).

$$\omega = 0.53 + 0.73(A_2 / A_1) \tag{3.1}$$

here, ω: weight per unit area on 1st floor, A_2: area of 2nd floor, A1: area of 1st floor

The structural frame consists of 8 I-shape panels (anchorage bolts ϕ 19), and 8 L-shape panels (anchorage bolts ϕ 19 and connection bolts ϕ 13) as shown in Fig. 3(b). The shear strengths of these panels are derived from Q_{su1}, and Q_{su2} in Eqn. 3.2. The lateral load carrying capacity Q_{su} is 498 (kN), in longitudinal direction and it deserves almost half of the total weight.

$$\begin{aligned} Q_{su1} &= 1.28 + 0.09N = 1.50\,(tf) \\ Q_{su2} &= 1.50 + 3.36 = 4.86\,(tf) \\ Q_{su} &= 1.50 \times 8 + 4.86 \times 8 = 498\,(kN) \end{aligned} \tag{3.2}$$

When hydrostatic pressure of inundation depth was assumed for tsunami load, base shear of this building is 1354 (kN) in Eqn. 2.1. This tsunami load is equivalent to 2.7 times of the frame strength, and the analytical result does not consistent with a fact that the building remained after tsunami. The drag force, which is calculated in Eqn. 2.2, is equivalent to 2.5 times of the frame strength, and it cannot demonstrate the observed damage as well.

The drag coefficient Cd in Eqn. 2.2 is actually an empirical value for a square pillar simply based on a relation between taking pressure area and measured loads in experimental studies. The drag force was also expressed as a summation of hydro pressure in front and behind the object. The past research measured flow velocity and inundation depth in front of and behind the object, and concluded that a drag force is equivalent to the hydrostatic pressure on both aspects as shown in Eqn. 3.3.

$$P_{f0} = 0.5\rho g\,(h_f{}^2 - h_b{}^2)B \tag{3.3}$$

In this case the inundation depth behind the object h_b is higher than 5.85 m because the building was completely under water. The wave height in front of the buildings can be supposed 7.5m at the maximum based on water traces on surrounding buildings. The drag force decreases up to 557 kN in above assumption. Another past research shows experimental study on drag force for a submerged square pillar, and tested a drag coefficient decreases up to 1.05 under super critical flow (in case Froude number is larger than 1.0). When inundation depth without building is assumed 5.0 m based on the water traces at inside of the surrounding building, the tsunami flow can be regarded as a super critical flow. In this case, a drag force on the building also decreases up to 664 kN in Eqn. 2.2. These two tsunami loads are equivalent to 1.1 ~1.3 times of the frame strength, and remaining of the building can be explained considering following factors, (1) the actual strength of the buildings might be generally higher than expected in the analysis, (2) observed flow velocity does not represented internal water flow.

3.2. Public building B

3.2.1. Outline of the building

The public building B is a three story reinforced concrete building structures built in 1970, and located in coastal area in Sendai city (Fig. 4). The out of plain deformation was occurred in outer frame by tsunami load. The damaged structural wall was two-story and four spans section without 2nd and 3rd floor slabs, in which columns and transverse beams are attached. The span length is 5.0 m in longitudinal direction. The aerial ratio of openings is 0.02, and the reduction of the tsunami loads for openings is negligible.

(a) ovewview of building B (b) Inside of public building B

Figure 4. Public three-story reinforced concrete building B in Sendai city

The reinforcement of columns, structural walls, transverse beams, and floor slab has been yielded. The concrete shows compressive failure at the bottom of the beams on the top floor. The maximum inundation depth was 10.5 m in front. The measured inundation depth was 6.78 m on side surface, and 5.0 m behind the structural wall (in the building).

Attached columns has 10-D25 rebar (SD345) in 700×900 mm section. The axial load on a column sections is roughly estimated 228 kN, and ultimate moment strength M_c is 655 kNm. 2-D22 rebar (SD345) is effective to out of plain bending moment strength for transverse beam. Width of the transvers beam is 400 mm, and out of plain moment strength M_{b2} is 93 kNm. Beams on the top floor has 5-D25 (SD345) top reinforcement in 400×800 mm section, and ultimate moment strength M_{b1} is 649 kNm. Double D10 rebar (SD295) are reinforced at 250 pitches in slab, and 88 top reinforcements are effective to the beam strength on the top floor. Triple D10 steel bars (SD295) are reinforced in 300 mm width wall and the number of steel bars is 68 (vertical) and 40 (horizontal) each for top, middle, and bottom reinforcement. The width of cover concretes is supposed to be 50 mm in above elements.

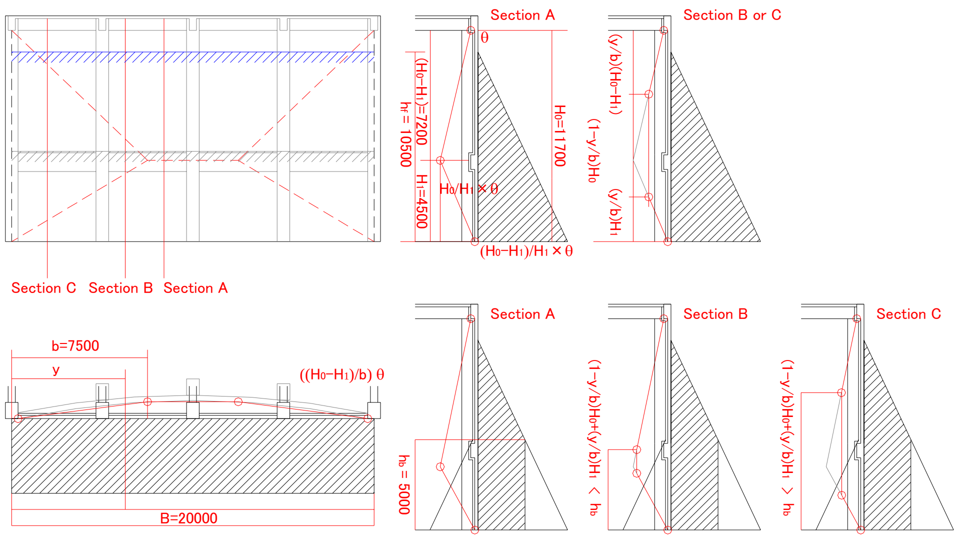


Figure 5. Plastic hinge mechanism on damaged frame

The analytical study was carried out with plastic hinge model on the damaged section. The total height of the structure H_0 is 11.7 m, and the most deformed point was 7.5 m (b = 7.5 m) inside from edges, and 4.5 m (H_1 = 4.5 m) height from ground level, which was derived from the survey of transit after tsunami. The folding line on damaged section was assumed as shown in Fig. 5. The accompanied floor slab and wall sections are fully considered in calculation of moment strength of beams and columns. These sections attached to columns and beams asymmetrically, and different moment strengths were adopted depending on rotating direction. The moment strength of each element are shown in Table 1. The required work W_0 for the plastic hinge mechanism is 20648×θ (kN m) with Eqn. 3.4 in assuming folding line and rotation angle θ at the top of the column.

Table 1. List of moment strength in plastic hinges

Location of Plastic Hinge	Tension in wall (slab)section	Compression in wall (slab)section	Index in Equation. 3.4	
Attached column	1965 kNm		M_C	
Beams on the top floor	1947 kNm		M_{LB}	
Transvers beam	93 kNm		M_{TB}	
Wall vertical reinforcement (40×3)	3215 kNm	595 kNm	M_{LW1}	M_{LW2}
Wall horizontal reinforcement (40×3)	683 kNm	350 kNm	M_{TW1}	M_{TW2}
Slab top reinforcement (88)	1319 kNm		M_{slab}	

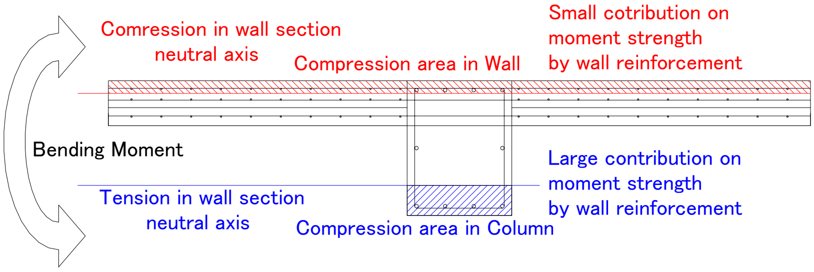


Figure 6. Calculation of out of plain bending moment strength for variable cross section

$$W_0 = (M_{LB} + M_{slab})\theta + (M_C + M_{LW1})\frac{(H_0 - H_1)}{H_1}\theta + (M_C + M_{LW2})\frac{H_0}{H_1}\theta \tag{3.4}$$
$$+ 2(M_{TB} + M_{TW1})\frac{(H_0 - H_1)}{b}\theta + 2(M_{TB} + M_{TW2})\frac{(H_0 - H_1)}{b}\theta$$

$$W_1 = \int_0^{B-2b}\left\{\begin{aligned}&\int_{H_0-h_f}^{H_0-H_1}(\rho g(z-(H_0-h_f))\times\theta z)dz\\ &+\int_{H_0-H_1}^{H_0}(\rho g(z-(H_0-h_f))\times(H_0-z)\frac{(H_0-H_1)}{H_1}\theta z)dz\end{aligned}\right\}dy \quad W_2 = 2\int_0^{b}\left\{\begin{aligned}&\int_{H_0-h_f}^{(H_0-H_1)\frac{y}{b}}(\rho g(z-(H_0-h_f))\theta z)dz\\ &+\int_{(H_0-H_1)\frac{y}{b}}^{(H_0-H_1\frac{y}{b})}(\rho g(z-(H_0-h_f))(H_0-H_1)\frac{y}{b}\theta)dz\\ &+\int_{(H_0-H_1\frac{y}{b})}^{H_0}(\rho g(z-(H_0-h_f))(H_0-z)\frac{(H_0-H_1)}{H_1}\theta)dz\end{aligned}\right\}dy \tag{3.5}$$

$$W_1 = \int_0^{B-2b}\left\{\begin{aligned}&\int_{H_0-h_f}^{H_0-h_b}(\rho g(z-(H_0-h_f))\times\theta z)dz\\ &+\int_{H_0-h_b}^{H_0-H_1}(\rho g(h_f-h_b)\times\theta z)dz\\ &+\int_{H_0-H_1}^{H_0}(\rho g(h_f-h_b)\times(H_0-z)\frac{(H_0-H_1)}{H_1}\theta z)dz\end{aligned}\right\}dy \quad W_2 = 2\int_0^{\frac{(H_0-h_b)}{(H_0-H_1)}b}\left\{\begin{aligned}&\int_{H_0-h_f}^{(H_0-H_1)\frac{y}{b}}(\rho g(z-(H_0-h_f))\theta z)dz\\ &+\int_{(H_0-H_1)\frac{y}{b}}^{(H_0-h_b)}(\rho g(z-(H_0-h_f))(H_0-H_1)\frac{y}{b}\theta)dz\\ &+\int_{(H_0-h_b)}^{(H_0-H_1\frac{y}{b})}(\rho g(h_b-h_f)(H_0-H_1)\frac{y}{b}\theta)dz\\ &+\int_{(H_0-H_1\frac{y}{b})}^{H_0}(\rho g(h_b-h_f)(H_0-z)\frac{(H_0-H_1)}{H_1}\theta)dz\end{aligned}\right\}dy \tag{3.6}$$
$$+2\int_{\frac{(H_0-h_b)}{(H_0-H_1)}b}^{b}\left\{\begin{aligned}&\int_{H_0-h_f}^{(H_0-h_b)}(\rho g(z-(H_0-h_f))\theta z)dz\\ &+\int_{(H_0-h_b)}^{(H_0-H_1)\frac{y}{b}}(\rho g(h_b-h_f)\theta z)dz\\ &+\int_{(H_0-H_1)\frac{y}{b}}^{(H_0-H_1\frac{y}{b})}(\rho g(h_b-h_f)(H_0-H_1)\frac{y}{b}\theta)dz\\ &+\int_{(H_0-H_1\frac{y}{b})}^{H_0}(\rho g(h_b-h_f)(H_0-z)\frac{(H_0-H_1)}{H_1}\theta)dz\end{aligned}\right\}dy$$

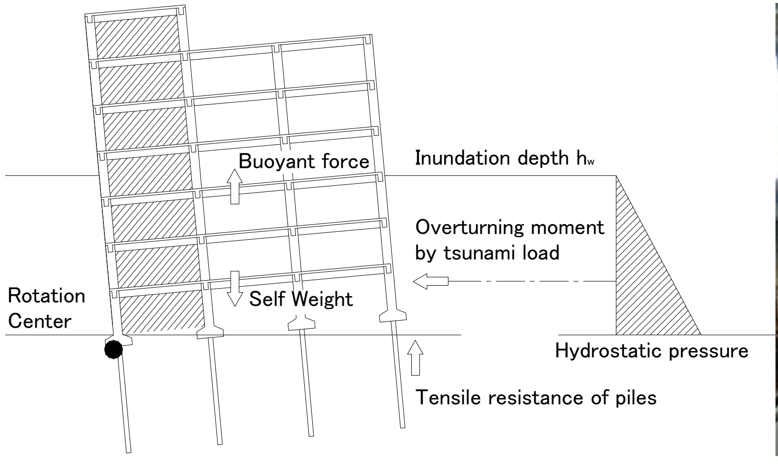
The hydrostatic force of inundation depth (h_f =10.5 m) is assumed for tsunami load in this model. Work by tsunami loads is 31101×θ (kN m), which is derived from integrating product of wave pressure and out of plain deformation in each point (a summation of W1 and W2 in Eqn. 3.5). This value is 1.5 times required work W_0 for the plastic hinge mechanism, so that analytical result does not consistent with observed damage. The mechanism strength seems to be equivalent to the external force because (1) obvious hinges was not observed at the bottom of columns, (2) steel reinforcement was not fractured on damaged frame. Therefore, another analytical model assumed a hydro static force of inundation depth behind the damaged frame (h_b = 5.0 m). Work by tsunami loads decreases down to 20177×θ (kN m) with a summation of W1 and W2 in Eqn. 3.6, and it is almost equivalent to the required work W_0 for the mechanism. This assumption can demonstrate observed damage in this building.

4. BUILDING OVERTUENING MECHANISM BY TSUNAMI LOAD

A ratio of overturning moment by tsunami loads to the moment resistance of the building was evaluated in following study. The distribution pattern of tsunami loads is assumed as a hydrostatic force based on the load distribution measured in experimental studies of tsunami. Wave pressure on openings is also ignored in the analysis. The rotation center is assumed at the bottom of footing in the overturning mechanism. The building resists to the overturning moment by both tensile force of piles, self-weight, and buoyant force (Fig. 7 (a)).

The buoyant force is basically equivalent to water weight of sinking building volume, if inflow water was ignored. However, this assumption overestimates the buoyant force generally. This paper proposed calculation of buoyant force considering inflow water. In remained buildings, water traces in the buildings were observed at the height of beams or hanging partitions (Fig. 7 (b)), and it indicates an existence of residual air space. In proposed model, the residual air volume on each floor is reflected in calculation of buoyant force. This model premises inflow water fulfill the building inside immediately, which is based on the fact that tsunami inundation depth increases gradually in most cases. Concrete density generally decreases in water, and weight per unit area of reinforced concrete buildings decreases down to 8.17 kN/m^2 (14 kN/m^2×(24-10)/24) including footings in calculation.

6-ϕ7 steel wires are used for a reinforcement of PC piles in overturned buildings. The fracture strength of PC steel wire (Type-A) is 58.3 kN, and the maximum tensile strength of a pile is 350 kN. The boring data in coastal area of Onagawa is shown in Table 2., and pull out strength of the pile R_{TC} is evaluated with Eqn. 4.1 based on AIJ design guideline. Estimate of pull out strength is 507 kN, which is larger than the fracture strength of steel wires.



(a) Overturning mechanism of the building (b) Inside of a survived building

Figure 7. Calculation of overturning moment, resisting moment, and buoyant force

Table 2. An example of boring site test in surrounding soil

Depth (m)	Soil Type	Density (ton/m^3)	SPT-N Value
1.3	Filled Gravel	2.10	31
2.3		2.10	42
3.3		2.10	9
4.3		2.10	46
5.3	Filled Sludge	1.50	1
6.3	Silt	1.75	2
7.3		1.75	2
8.3		1.75	2
9.3	Clay with Gravel	1.50	19
10.3		1.50	18
11.3		1.50	19
12.3		1.50	38
13.3		1.50	40

$$R_{TC} = (\sum \tau_{ST} L_S + \sum \tau_{CT} L_C)\varphi + W_p \quad (kN) \tag{4.1}$$

Here, τ_{ST}: friction stress on peripheral surface of piles in sand layer (kN/m^2) (≒ 2N), τ_{CT}: friction stress on peripheral surface of piles in clay layer (kN/m^2) (≒5N/0.8), Ls: width of sand soil layer (m), Lc: width of clay soil layer (m), φ: peripheral length of pile (m), W_p: pile weight (kN), N: Average SPT-N Value in each soil layer

5. COMPARISON BETWEEN OVERTUNING MOMENT AND MOMENT RESISTANCE

Accommodation building C is four-story reinforced concrete building with pile foundation located 200m away from Onagawa coast (Fig. 8.). The floor plan is 4×6 m, and the top floor has 1m setback in transvers direction. Total height of the building is 12 m. The building overturned in transverse direction by tsunami loads, and moved 70 m from original position. The trace of inundation depth was observed at the 15 m height in front of a proximate building, while the clear distinction between water traces was not observed in front and behind the building. 12 PC piles seem to be effective to resist overturning moment out of 32 piles under footings. Each pile has 6-ϕ7 steel rebar in 300 mm hollow round section. The tensile resistance of piles was obtained from yielding force of steel bars, which was smaller than pull out strength based on the boring site test in surrounding soil. The weight per unit area of this building decreases from general reinforced concrete buildings for submergence.

Figure 8. Overview of overturned four-story reinforced concrete building C with pile foundation

The aerial opening ratio in front elevation is 0.052.The total weight of the structures W_b is 931 kN in water. Buoyant force of the building F_v is 928 kN, which is equivalent to the water weight of residual air space (4F: 15.3 m^3, 3F 18.0 m^3, 2F 15.6 m^3, 1F 27.6 m^3, BF 18.0 m^3), and self-weight is almost cancelled with buoyant force. The tensile resistance of a pile is 350 kN. The moment resistance by a summation of contribution for self-weight and piles are 14706 kN m as shown in Eqn. 5.2, in which the distance from rotation center to center of tensile piles is supposed to be 3.5 m. On the other hand, the overturning moment M_h by hydrostatic tsunami loads is 28108 kNm at 15 m height of tsunami, which is derived from Eqn. 5.1. The moment resistance M_b is 52 percent of the overturning moment M_h.

Past research reported on the shear strength of PC pile without axial load, and average ultimate shear stress is 0.85 N/mm^2 in this study. It means a summation of pile shear strength is 1568 kN, while the lateral tsunami loads is 6023 kN in Eqn. 2.1. The sum of pile shear strength is about one-fourth of lateral tsunami loads. Above analytical results are consistent with overturning of the building.

$$M_h = \int_0^{h0} (\rho g z (h_w - z) \times B(1-\zeta))\, dz \quad (kNm) \tag{5.1}$$

$$M_b = (W_b - F_v) \times 2.0 + F_p \times 3.5 \qquad (kNm) \tag{5.2}$$

6. CONCLUDING REMARKS

The maximum inundation depth of tsunami in several regions on 2011 Great Tohoku Earthquake exceeded 10 m, and water depth tends to increase gradually in these areas. Two remaining and damaged reinforced concrete buildings structures are analysed, which were built in one of above regions. One-side hydrostatic pressure of inundation depth does not consistent with observed damage, and acting force on these building structure can be explained considering hydro pressure behind the building in related concept with drag force on object (a summation of hydro pressure in front and behind the object). The moment resistance of overturned four-story reinforced concrete building with pile foundation is evaluated with detail analysis model considering piles and buoyant force. The capacity for overturning or drift of base foundation was not enough for the tsunami loads, and the analysis can demonstrate the observed damage of the building.

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