SEMARAK ILMU PUBLISHING
202103268166(003316878-P)

Journal of Advanced Research in Applied Sciences and Engineering Technology

Journal homepage:
https://semarakilmu.com.my/journals/index.php/applied_sciences_eng_tech/index
ISSN: 2462-1943

Capacity Analysis of Reinforced Concrete and Composite Concrete with the Nonlinear Time History of Imperial Valley Earthquakes

Anthony Costa[1,*], Thania Andini[1], Wadirin[2], Dendy Adanta[3], Sakura Yulia Iryani[1], Kencana Verawati[4]

[1] Department of Civil Engineering, Faculty of Engineering, Universitas Sriwijaya, Palembang-30128, South Sumatera, Indonesia
[2] Department of Mechanical Engineering Education, Faculty of Teacher Training and Education, Universitas Sriwijaya, Palembang-30128, South Sumatera, Indonesia
[3] Department of Mechanical Engineering, Faculty of Engineering, Universitas Sriwijaya, Palembang-30128, South Sumatera, Indonesia
[4] Department of Transportation, Faculty of Engineering, Universitas Negeri Jakarta, Rawamangun-13220, East-Jakarta, Indonesia

ARTICLE INFO

Keywords:

Intake building; ETABS; nonlinear time history

ABSTRACT

The design of a water intake system will be a highly efficient water use plan for the future fulfilment of water resources. In the case of pump intake building, a pier-type intake with reinforced concrete, steel structures and composite steel piles was used. 3D finite element modelling with a nonlinear time history loading method using Finite Element Analysis software was conducted to determine the failure performance of the structural capacity. The ground motion data utilised were Imperial Valley-06 of 1979, which were matched with the response spectrum of Bengkulu City and entered into the programme as earthquake load scaled to 2 times the original scale. The results of the structural capacity analysis included floor deviation, beam deflection, composite ratio and steel frame ratio. When the earthquake scale was increased to 1.5 × A0g, structural damage occurred in the steel columns because the damage ratio value was ≥1, indicating that the steel structure's load value exceeded the steel material's stress–strain capacity. The reinforced concrete pile cap beams also showed evidence of structural damage due to increased seismic loads because the deflection value of the beam structure exceeded the predetermined allowable deflection value. However, no structural failure occurred in the composite column structure because the capacity of the composite material was still able to withstand the earthquake's magnitude.

1. Introduction

Water is a natural resource that is the main need to support the survival of humans every day. As the growth of humanity increases in an area, the clean water needed also increases. The imbalance between the need for clean water and the availability of water resources can hamper human survival, such as in the food security and irrigation sectors. In case the availability of clean water cannot meet the water needs of the population in the area, a crisis of clean water, decreased productivity levels and an impact on people's health will occur. Therefore, planning an intake structure that produces water is necessary to achieve high efficiency in meeting water resource needs in future.

* *Corresponding author.*
E-mail address: anthonycosta@ft.unsri.ac.id

https://doi.org/10.37934/araset.63.2.5463

To supply clean water to residents' settlements in Karanganyar, Pulokerto, Regional Drinking Water Company (PDAM) plans to construct an intake building with raw water sources from the Musi River. The intake building is a clean water supply system facility in the form of a raw water intake building established on the side of a water source to go through the next processing stage. It uses a wharf intake with the water source coming from the Musi River water flow. The choice of this type of intake is due to the differences in water levels during the tidal season. It will be highly economical because the volume of excavation and embankment work required is not too large. The main intake building structure uses reinforced concrete with composite steel piles. Composite structures are selected because composite piles can minimise the influence of external loads, have a high bearing capacity and are plastic against earthquake loads [1]. In addition, composite piles are more effective for buildings in river areas because precast piles have a lower bearing capacity and sink faster [2].

Structural failure performance is assessed to review and evaluate building performance against structural capacity on the basis of SNI 1726:2019. In determining the capacity of concrete and steel composite structures in a construction building, earthquake forces are given as irregular forces with random durations on the structure. Nonlinear time history is an analysis method for determining the dynamic response of structures that behave linearly or nonlinearly to ground motion. In this study, time history earthquake loads were used and matched with the spectral response of an area to be input into the programme as earthquake loads. The earthquake load was increased on the scale of the earthquake with a gradual multiplier factor (levelling) until a failure was identified in the structure to determine the structural capacity. The results included structural failure behaviour in the form of lateral drift (drift), composite ratio and beam deflection, which were analysed on the basis of SNI 1726:2019 [3]. Time history analysis, also known as nonlinear dynamic analysis, is an optimal method for evaluating the seismic behaviour of structures. It is the direct solution of the differential equations of an Multi Degree Of Freedom (MDOF) system subjected to a time history of accelerations representative of an earthquake expected to occur [4]. On this basis, this research aimed to determine and analyse the effectiveness and capacity of an intake wharf structure as a water source building through 3D numerical programme modelling with additional loads in the form of Imperial Valley time history earthquake loads on the response spectrum of Bengkulu.

2. Case Study

2.1 Wharf Intake

In this case study, the intake building is planned with a wharf intake type. The selection of the wharf type is influenced by the factors of needs served in the form of raw water sources from rivers, river depth, river width and river discharge factors. In addition, the selection of the intake type is influenced by the differences in water levels during the tidal season. The raw water comes from rivers, which will be considerably economical because it does not require large volumes of excavation and stockpiling. Composite piles are used as the foundation with SD soil site class in medium soil.

The intake structure must be located outside the helicoidal flow bend or the spiral movement of river water, which causes erosion outside the river and siltation of the inside [5]. When the river level is high, sediment transport will approach the intake structure and flow to the bend away from the intake. This phenomenon can lead to the formation of sediment-free areas at the edge of the intake and facilitate the tapping process of the raw water flow.

2.2 Composite Structure

A composite structure is a structure formed from a combination of materials with different characteristics to work together to withstand loads. The commonly used composite structure is reinforced concrete, a combination of plain concrete and steel reinforcement or profile steel. Plain concrete has the characteristic of high compressive strength but low tensile strength. Nevertheless, profile steel has a high tensile strength to provide that required by plain concrete. These two materials can be combined into a composite structure with better characteristics than those of the forming materials. The configuration between the concrete and steel profile will withstand the forces acting on a structure with the compressive forces retained by the concrete and the tensile forces retained by the steel. The advantages of composites are steel weight savings, lower steel beam cross sections, increased floor stiffness, less span length required and increased load-bearing capacity. Three types of failure modes are experienced by composite structures, namely, failure in shear in compression reinforcement (mode 1), bending failure in tensile reinforcement (mode 2) and failure in joints (mode 3) [6,7]. Damage to the composite column structure is indicated by the occurrence of damage to the structural components caused by exceeding the stress–strain value of the constituent materials due to excessive loads [8].

2.3 Non-Linear Time History Analysis

Dynamic analysis is an analytical method for identifying structural responses to nonlinear earthquake loading. The planned structure is elastic and does not experience changes in mass, damping or stiffness. However, after exceeding its elastic capacity, a nonlinear structure will experience mass, damping and stiffness changes when subjected to an inelastic response.

The dynamic analysis method of earthquake acceleration time history can be used in the structural analysis of earthquakes. Time history response analysis examines structural dynamics, in which a mathematical model of the structure is subjected to the time history of recorded earthquakes or mock earthquakes against the time history of the specified structural response to determine if it will withstand the forces acting on the structure. The compressive forces are retained by the concrete and the tensile forces are retained by the steel. The advantages of composites are steel weight savings, lower steel beam cross sections, increased floor stiffness, less span length required and increased load-bearing capacity [9].

3. Material and Methods

In this research, a computer model was prepared in three dimensions and structural analysis was performed. Using computer models and analysing such models is a reliable and appropriate approach in structural analysis to determine the extent of damage to buildings. This computer-dimensional modelling is called 3D finite element model [10]. The finite element method numerically models a structure in the form of 3D elements using the ETABS programme. The analysis results will be displayed as deformation or deflection, pressure value, strain value and damage ratio [11]. This research used the quantitative analysis method via the ETABS programme based on finite element model analysis. The design of the intake building included steel, reinforced concrete and composite concrete with the quality of the concrete material used, namely, f'_c of 24.9 MPa, profile steel quality of 400 MPa and reinforcing steel qualities f_y of 420 and 280 MPa.

3.1 Finite Element Modelling

The dimensions of the intake structure were two floors, a total height of 7 m and a building length of 24,15 m. The pile cap beam size was 0.6 m × 0.65 m, the IWF column size was 150 × 300 and the composite pile foundation had a diameter of 60 cm and a depth of 8 m. The research method used was a case study, which reviews, analyses and compares an object of research. This research was conducted at the Karanganyar intake building project, Pulokerto, with reinforced concrete beams and composite steel piles as research objects. The structure was analysed by using Finite Element Analysis Software via the ETABS with the nonlinear time history analysis method [12].

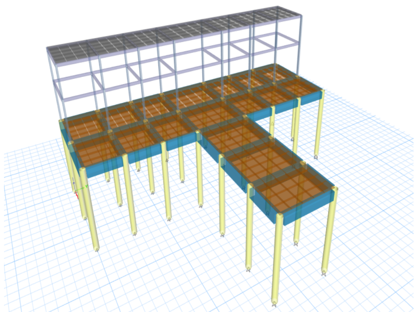

Fig. 1. 3 Dimension modelling view

The research stages are depicted in the following flowchart in Figure 2.

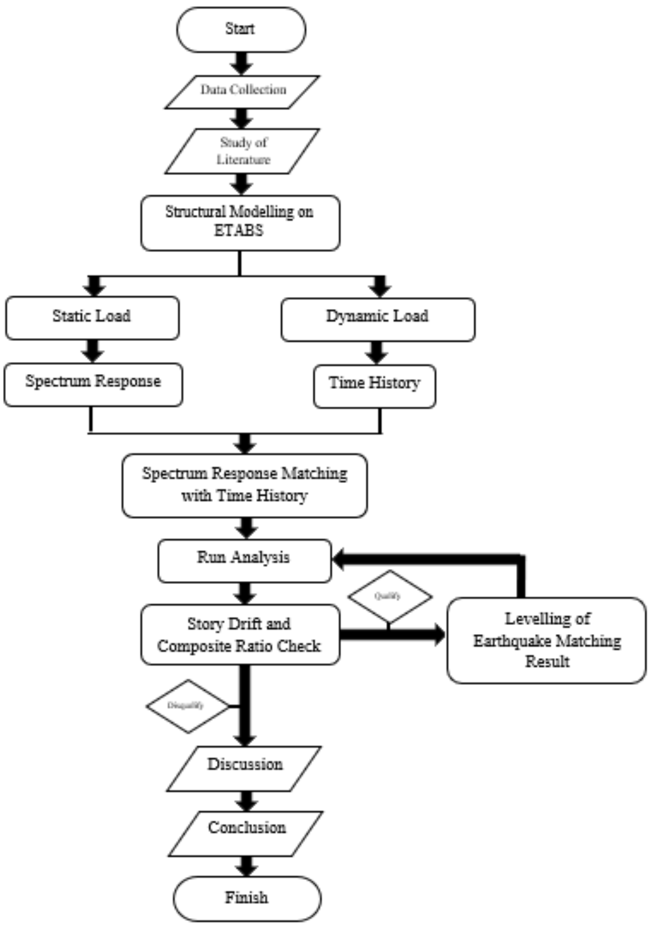

Fig. 2. Research Flowchart

3.2 Assessment Loading and Time History of Earthquake Loads

In the time history process, the variables change in nonlinear and linear evaluations to assess the seismic demonstration of constructions [13]. In planning earthquake-resistant building structures on the basis of SNI 1726-2019, the load classification consists of dead loads, additional dead loads, live loads, water loads, soil loads and earthquake loads [14]. The earthquake load input in the intake structure modelling was the spectrum of the earthquake load response for the Bengkulu region in the medium soil category (SD). In 2007, an earthquake with a magnitude of 8,7 occurred in the province of Bengkulu, Indonesia. The interpretation of the damage caused by this earthquake indicated a scale of 9, meaning that the seismic load caused severe damage to the structural frame and foundation of the building, resulting in the building collapsing [15]. The response spectrum was adjusted to the Imperial Valley-06 (1979) time history data obtained from PEER Berkeley, with the direction of seismic loads along the x and y axes. Imperial Valley time history earthquakes are commonly used in the analysis of seismic load structures because they have an adequately large magnitude, namely, 6,53 SR and a failure distance of 0,56 km; thus, they exert a considerable influence on structures.

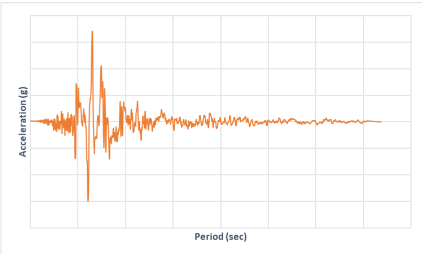

Fig. 3. Imperial valley time history (X direction)

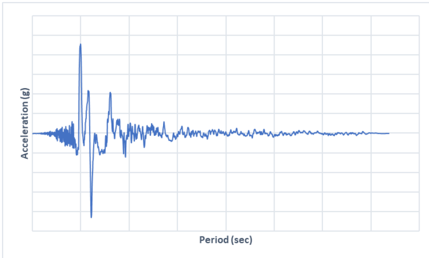

Fig. 4. Imperial valley time history (Y direction)

All components and structural elements were designed with a strength that equals or exceeds the effect of factored loads with the load combinations in Table 1 on the basis of SNI 1726-2019. For earthquake loads in the x and y directions, the load combination was a spectrum response that has been matched with the time history of Imperial Valley.

Table 1
Load combinations

Load	Combo1	Combo2	Combo3	Combo4	Combo5	Combo6	Combo7
Dead	1,4	1,2	1,2	1,2	0,9	1,2	0,9
Live	-	1,6	1,0	1,0	-	1,0	-
Water	-	-	1,2	-	-	1,2	0,9
Soil	-	-	-	0,9	-	0,9	0,9
Earthquake Load (X)	-	-	-	-	-	1,0	-1,0
Earthquake Load (Y)	-	-	-	-	-	1,0	1,0

4. Result and Discussion

4.1 Matching Earthquake Loads

The Imperial Valley time history earthquake data matched with the Bengkulu response spectrum are used to obtain an approach to time history data at the research location. The matching process is carried out using the ETABS and Seismomatch programmes. The matching result from ETABS is in the form of a time history plot that resembles the response spectrum, as shown in Figure 5 and Figure

6 and the matching result from Seismomatch is in the form of the maximum acceleration of the earthquake. The method used is probabilistic seismic hazard analysis (PSHA) by considering the seismic hazard at a location within a certain period of time; the variables that determine the PSHA assessment are the earthquake zone, earthquake magnitude and the earthquake distance from a location [16].

The maximum earthquake acceleration in the Imperial Valley time history is 0,792g and the Bengkulu spectrum response is 1,0g. After matching, the maximum earthquake acceleration in the x direction is 1,112g in a period of 0,54s. The maximum earthquake acceleration in the Imperial Valley time history is 0,811g and the Bengkulu spectrum response is 1,0g. The maximum earthquake acceleration in the y direction is 1,165g in a period of 0,36s. Whilst generating spectrum-matched time histories, practitioners only focus on matching a target response spectrum in terms of spectral accelerations. To minimise the biased estimate of seismic response analysis, spectrum-matched time histories should meet predefined tolerance to reduce the spectral dispersion with a relatively small tolerance of less than 10% [17].

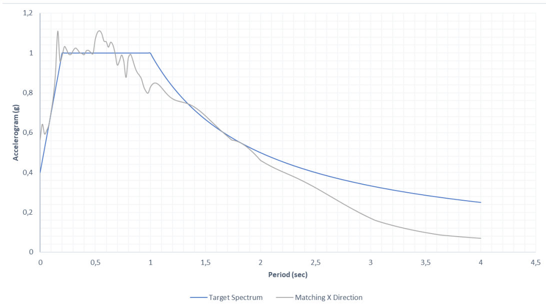

Fig. 5. Matching imperial valley time history (X direction)

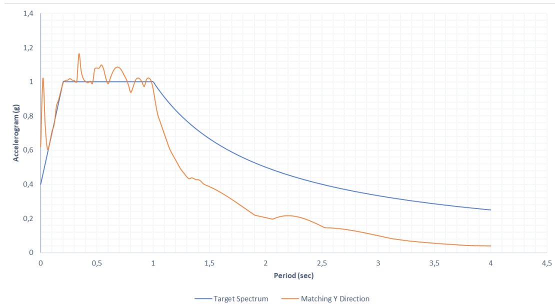

Fig. 6. Matching imperial valley time history (Y direction)

4.2 Structure Failure Capacity Analysis

Damage status, commonly called performance-based design in a building structure, is needed as a form of structural performance assessment of a design evaluation; some studies focus on deflection capacity for various structural damage limits. In other studies, the drift ratio value of a structural element's eccentricity is caused by the occurrence of an axial load, which results in damage to the column structural element starting with the occurrence of flexural cracks [18]. Capacity analysis, as a benchmark for structural failure, refers to four assessments: floor drift, beam deflection, steel ratio and composite ratio. The four assessments are categorised as failed if they exceed the permit requirements determined on the basis of SNI 1729-2002 [19].

4.3 Floor Drift

Floor drift is the behaviour of a structure when subjected to loads, especially earthquake loads [20]. The output of the floor deviation is the joint displacement of the structure in the x and y directions from the initial coordinates of the building [21]. Floor drift results from a combination of envelope loading in the x and y directions from the starting point of the building with an increase in the earthquake scale with multipliers of $1 \times A_o g$, $1.5 \times A_o g$ and $2 \times A_o g$.

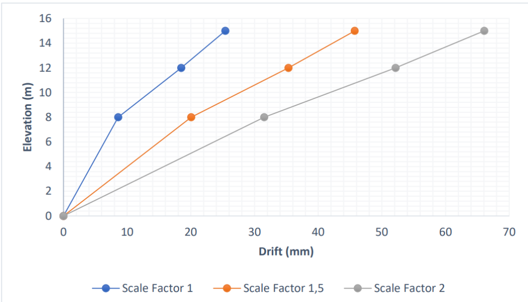

Fig. 7. Floor drift graph (X direction)

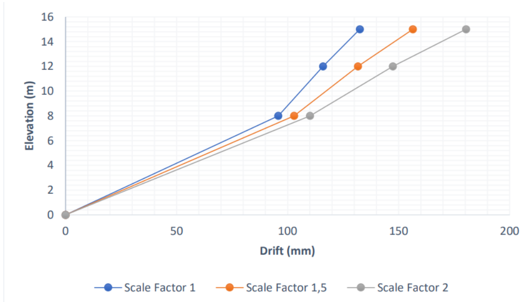

Fig. 8. Floor drift graph (Y direction)

The floor deviations in the x and y directions are listed in Table 2.

Table 2
Structure story drift with scale factor

Scale Factor	Story	Story Drift (mm)	
		X Direction	Y Direction
1 × Aog	3	25,403	132,505
	2	18,479	115,966
	1	8,598	95,74
2 × Aog	3	45,734	156,422
	2	35,315	131,664
	1	20,053	102,891
3 × Aog	3	66,062	180,337
	2	52,15	147,361
	1	31,506	110,041

According to Table 2, the structure fails even though the earthquake has no multiplier factor. The deviation values do not meet the deviation permit requirements for each floor. The reason is that the earthquake's maximum acceleration occurs faster than that in the x direction, which is 1.165 g in a period of 0.36 s.

4.4 Beam Deflection

The deflection on the beam (Ln) in terms of the location of the maximum deflection on each span is 4m, 5m and 6 m. The maximum deflection is analysed with an Ln/240 mm deflection permit based on SNI 1729-2002 Article 6.4 [22]. Three deflection failure categories represent structural damage based on deflection and are combined with 4 target damage conditions (elastic limit with minimal damage, elastoplastic with moderate damage and total failure due to the end limit) [23]. Beams that produce maximum deflection and exceed the permit limits do not meet safety requirements and fail [24]. The maximum deflection of the beam is presented in Table 3.

Table 3
Recapitulation of the location and value of the maximum deflection on the beam

Scale Factor	Length (m)	Permitted Deflection (mm)	Positions	Structure Deflection (mm)	Status (OK/Not OK)
1 x Aog	4	16,667	B7	1,194	OK
	5	20,833	B43	0,919	OK
	6	25	B32	1,015	OK
1,5 x Aog	4	16,667	B7	1,328	OK
	5	20,833	B43	1,182	OK
	6	25	B32	1,058	OK
2 x Aog	4	16,667	B7	1,436	OK
	5	20,833	B43	1,446	OK
	6	25	B32	1,107	OK

In Table 3, reinforced concrete beams resist earthquake loads because they still meet the deflection permit requirements. This is caused by the stiffness retained on the concrete and bending elements contained by the reinforcing steel, so the resulting Deflection is still safe. The location of the maximum Deflection on the beam can be seen in Figure 9 as follows.

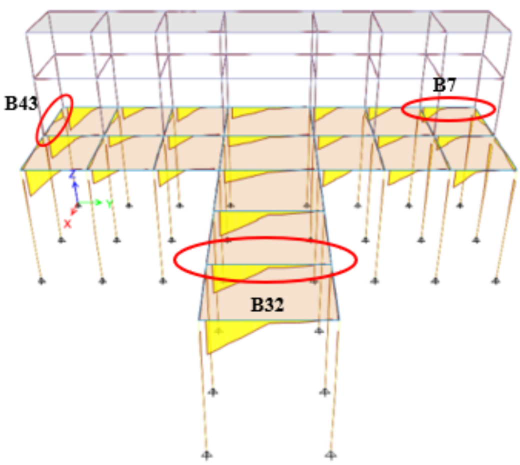

Fig. 9. Maximum deflection location

4.4 Steel Ratio and Composite Ratio

An analysis is carried out on all load combinations to the maximum load output. If the steel frame ratio value is less than 1, the design check applies; thus, the steel profile is still in the safe category. On the contrary, if the value of the steel frame ratio is more than 1, the steel structural profile is declared to have failed and requires replacement and rechecking [25].

Meanwhile, design checks apply when the value of the composite column ratio is less than 1 to determine the value of the failure of composite pile elements. In composite columns, structural failure occurs because the axial strength of the column is affected by local buckling [26]. Nevertheless, the structural profile is still in the safe category. If the value of the composite column ratio is more than 1, the composite structural profile is declared to have failed and requires replacement and rechecking.

Table 4
Recapitulation of structure failure on steel ratio

Scale Factor	Location	Structure of Steel Ratio	Permitted Steel Ratio	Status (OK/Not OK)
1 x Aog	C4	0,927	1	OK
	C5	0,939	1	OK
1,5 x Aog	C4	1,014	1	OK
	C5	1,028	1	OK
2 x Aog	C4	1,102	1	OK
	C5	1,118	1	OK

Table 5
Recapitulation of structure failure on composite ratio

Scale Factor	Location	Structure of Steel Ratio	Permitted Steel Ratio	Status (OK/Not OK)
1 x Aog	C10	0,732	1	OK
	C15	0,728	1	OK
1,5 x Aog	C10	0,848	1	OK
	C15	0,841	1	OK
2 x Aog	C10	0,965	1	OK
	C15	0,954	1	OK

5. Conclusion

A structural failure occurred in the intake 3D finite element model because of the value of the structural deviation at the floor exceeding the deviation permission requirement and exceeding the stress–strain capacity of the material. When the earthquake scale was increased to 1.5 × A_0g, structural damage occurred in the steel columns owing to the structural damage ratio value ≥ 1. That is, the load value received by the steel structure exceeded the stress–strain capacity of the steel material itself. In addition, indications of structural damage due to increased earthquake loads occurred in the reinforced concrete pile cap beams because the deflection value of the beam structure exceeded the predetermined deflection allowance. Nevertheless, for the composite column structure, no structural failure occurred because the capacity of the composite material was still able to withstand the magnitude of the load that occurred in the structure.

Acknowledgement
This research was not funded by any grant.

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