Assessing the Performance of a Transportation Lifeline in the Philippines, the Light Rail Transit (LRT) System, Under a Large Magnitude Earthquake

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Summary

This paper therefore investigates the reliability index of the columns of the LRT under a Level 1 (El Centro) earthquake andLevel 2 (Tohoku-Kanto) earthquake using ordinary Monte Carlo Simulation. Based from the maiden structural plans of LRT, the slenderness ratio of columns based from the ACI 318 was observed and checked for buckling failure.The reliability indices of the light railway transit, specifically in one of its reinforced concrete pier, is 3.06 (unconfined, NSCP 2010) and 3.67 (confined, NSCP 2001) when it was simulated under a Tohoku-Kanto Earthquake. A similar scenario was also computed for the simulation of El Centro Earthquake, that is, 3.50 (unconfined, NSCP 2010) and 4.10 (confined, NSCP 2010). This can be attributed to the effectiveness of the confinement model used in this simulation, that is, a maximum of 92% improvement of confinement in the reinforced concrete pier.

Keywords:light rail transit system, Tohoku-Kanto earthquake, reliability index, Monte Carlo simulation

Introduction

The Philippines' capital has its mass transit, the Light Rail Transit System (LRT), constructed in the 1980s as part of the government's modernization efforts in the field of transportation. Over the past thirty years the LRT has withstood a number of natural hazards including a strong earthquake in July of 1990. Due to this event, the Philippine government initiated the earthquake reconstruction project and made recommendations to retrofit important bridges.

The assessment of a mass railway transit is to ensure its serviceability despite of the absence of significant structural retrofitting. By using reliability study [3], it is the first of its kind in the country. This paper aims to conduct primarily an assessment of a transportation lifeline in Manila through the applicability of a method known as reliability index by ordinary monte carlo simulation. Specifically, a typical pier of the light railway system was subjected to seismic forces brought by the ground acceleration of Level 1 (El Centro Earthquake) and Level 2 (Tohoku-Kanto Earthquake) magnitudes, and analyzed as a single degree of freedom, lumped mass model using Newmark Beta Method to compute for the structural response.

Methodology

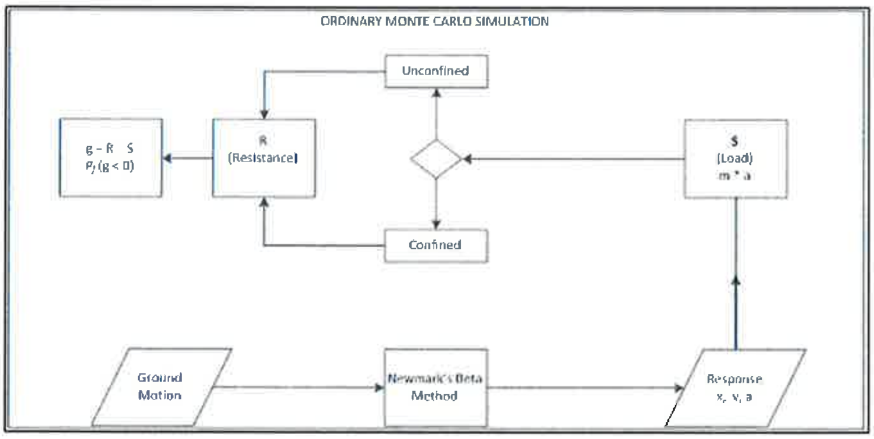

Figure 1. Flow of Methodology

Based from Figure 1, ground motion acceleration from two levels of earthquake magnitudes, i.e. the El Centro of 1940 and Tohoku-Kanto of 2011 were the inputs in a MatLab script that computes the structural response of single-degree-of-freedom lumped mass model of a typical pier of the LRT. This typical reinforced concrete column was based from the maiden plans of LRT, thirty years ago, as reproduced from Figures 2 and 3. The computed properties which were used to compute necessary parameters later, were summarized in Table 1. It can be observed from this structural plan the confinement of the rebars.

Table 1. Summary of Computed Parameters of Concrete Pier

weight density (kN/m3)	plan area (m2)	height (m)	volume (m3)	weight (kN)	mass (kg)	stiffness (kN/m)	Elastic Modulus (MPa)
24	2.38	7.75	18.445	442.7	8165232.6	366.2	24781

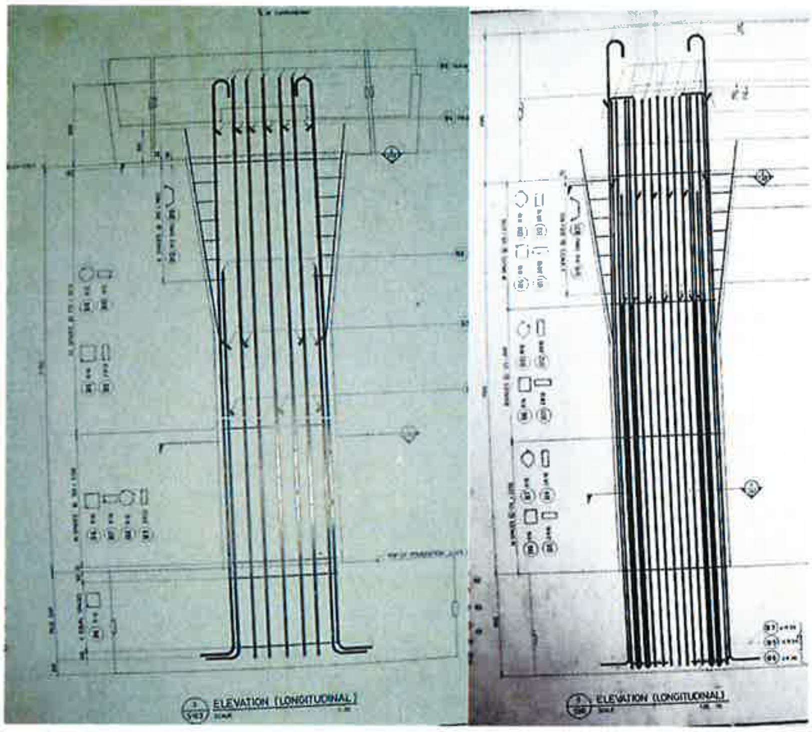

Figure 2. A Typical Elevation View of LRT 1 Pier

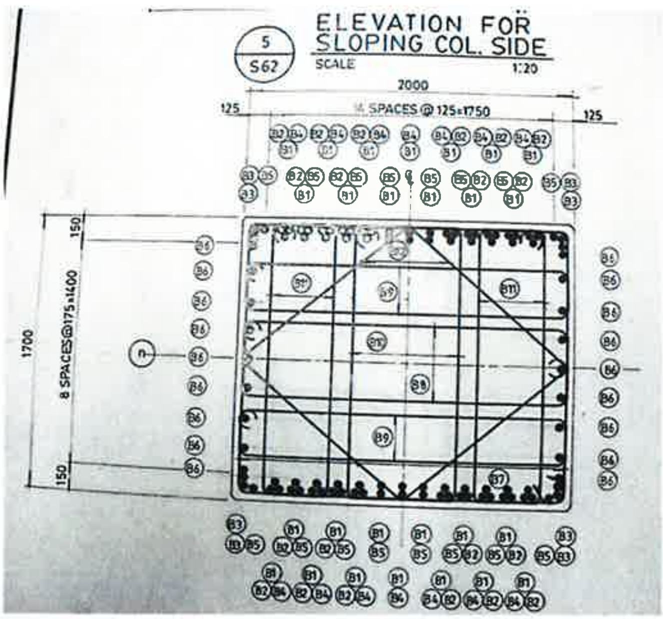

Figure3. A Typical Section View of LRT 1 Pier

Referring to the transcription of the structural plans, the compressive strength of unconfined concrete, fco, is 280 kg/cm^2 (27.468 MPa) and transverse steel yield strength, f_{yh}, is 2800 kg/cm^2 (274.68 MPa). Based from these data, a linear interpolation was used to compute for the corresponding parameters of variables —see Table 3— in accordance to Table 2 [2]. The mass of the concrete pier of 8,165,232.6 kg was computed based from a unit weight of 24 kN/m^3 and a dimension of 1.4 m x 1.7 m x 7.75 m of the pier. A volume of 18.445 m^3 of concrete alone was computed. Based from these, the computed weight of the concrete pier was 442.68 kN and from the NSCP 2010 computation of concrete modulus of elasticity, a 24,781 MPa was computed. The larger moment of inertia was calculated to be 0.5731 m^4.

Table 2. Statistical parameters of material properties and dimensions

Property	Mean value	coeff.of variation
Concrete compressive strength		
fc' = 3 ksi	2.760 ksi	0.18
fc' = 4 ksi	3.390 ksi	0.18
fc' = 5 ksi	4.028 ksi	0.15

1 ksi = 6900 Pa; 1 in = 25.4 mm
A range of values is presented in some instances because data from multiple sources were used.
Source: Adapted fromEllingwood, Galambos,

After obtaining the structural response, specifically the acceleration, it is now a parameter used to the inertial force as the Load function, **S**. It is noted that the structural response is probabilistic in nature, as it can be reflected from Table 3 that summarizes the parameters of variables used in the ordinary MCS. The Load function is expressed in one of the NSCP 2001 Section 409.3.3 and NSCP 2010 Section 409.3 factored loads [21], i.e. (where D is the dead load, and E for seismic load)

$$S = 0.99D + 1.1E \tag{1a}$$

$$S = 0.90D + 1.0E + 1.6H \tag{1b}$$

This Load Function,**S**, is then compared to the Resistance Function, **R**. The resistance (a.k.a. capacity) function by nature is probabilistic (refer to Table 3) with parameters divided into two sets: the unconfined and confined strength of the reinforced concrete column [6]. The unconfined model used was based from [7]. For the purposes of comparison of unconfined and confined concrete columns, it can be shown in Figure 4 that significant change in the stress-strain diagram occurs.

Table3. Summary of parameters of variables

Variable	Distribution	Mean	Standard Deviation
RESISTANCE			
f_{co}	Lognormal	23.734 MPa	4.272 MPa
f_{yh}	Lognormal	312.33 MPa	36.542 MPa
ρ_s	Lognormal	1.171E-03	1E-06
LOAD			
$^{1}P_{EQ}$	Lognormal	4580 kN	4001 kN
$^{2}P_{EQ}$	Lognormal	2442 kN	2440 kN

[1]Using Newmark Beta Method for Tohoku-Kanto Earthquake
[2]Using Newmark Beta Method for El Centro Earthquake

Figure4. Stress-Strain Diagram of Confined and Unconfined Concrete Column. (Source: Miller, 2006)

The confinement model used was based from the research of Sakai in 2001 [1], [10].

$$f'_{cc} = f_{co}'(0.94 + 4.7C) \tag{2}$$

where:

$$C = K_S\left[\frac{\rho_s f_{yh}}{2f_{co}'}\right] \tag{3}$$

$$K_s = \left[1 - \frac{s}{d\ tan30°}\right] \geq 0 \tag{4}$$

For non-prestressed members with existing steel-tie reinforcement, the nominal compressive force is calculated as (from [7]):

$$P_n = 0.80\left[0.85 f_{cc}\left(A_g - A_{st}\right) + f_{yh}A_{st}\right] \tag{5}$$

where:

f_{co}= the concrete's specified strength
f_{cc}= the confined concrete strength
ρ_s= the steel ratio, A_{st} / bh
b,h = pier's cross-sectional area dimensions
s=confinement ties spacing
d=the pier's effective depth
f_{yh}= the steel longitudinal reinforcement yield strength

The resistance function variables used are summarized in Table 4. Using the performance function, **g**, i.e. the difference of resistance function to the load function, the probability of failure is now calculated using the formula in [2].

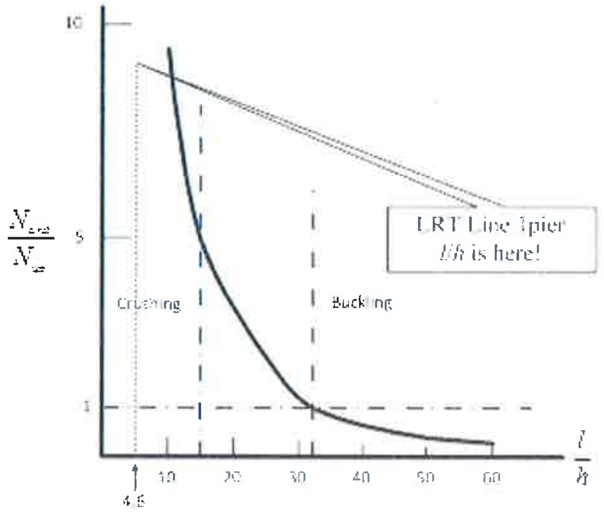

Figure5. The graph of the ratio of critical load to the crushing load versus the length to width ratio. (Source: Miller, 2006)

Buckling is a critical issue for structural stability in structural design[4], [14], [15]. In most of the buckling analyses, applied loads, structural and material properties are considered certain. But in the case of this research, the ratio of the unsupported length, *l*, to the largest dimension, *h*, is less than a critical value of 15, that is, it is believed that the column pier will fail in crushing. See Figure 5.

Results

Based from the ordinary MCS using a MatLab script, a summary of the results can be seen from Table 4, showing only the 1,000,000 iterations.

Table 4. Summary of Results of Ordinary Monte Carlo Simulation

	Tohoku-Kanto Earthquake 2011			El Centro Earthquake of 1940		
	Unconfined	Confined	% diff	Unconfined	Confined	% diff
	Based from NSCP 2001					
P_f	0.00146	0.00012	-92%	0.00029	0.00013	-55%
β	2.97	3.67	0	3.44	3.65	6%
	Based from NSCP 2010					
P_f	0.00109	0.00039	-64%	0.00023	0.00002	-91%
β	3.06	3.34	9%	3.5	4.11	17%
% P_f	-0.00037	0.00027		-0.00006	-0.00011	

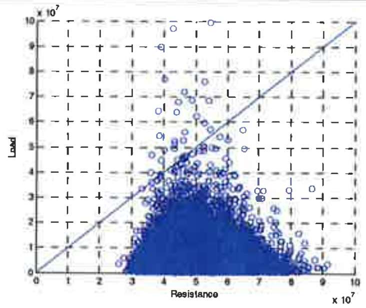

Figure 6. Result of MCS (Unconfined, El Centro, NSCP 2001): Pf=0.000290, β=3.440799

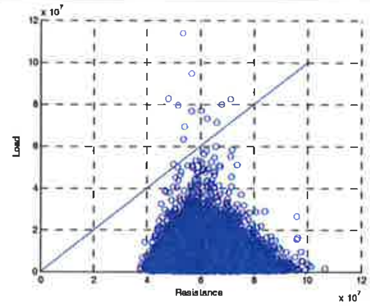

Figure 7. Result of MCS (Confined, El Centro, NSCP 2001): Pf=0.000130, β=3.652203

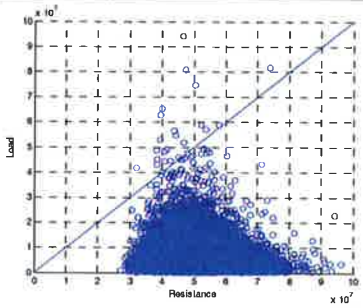

Figure 8. Result of MCS (Unconfined, El Centro, NSCP 2010): Pf=0.000230, β=3.503029

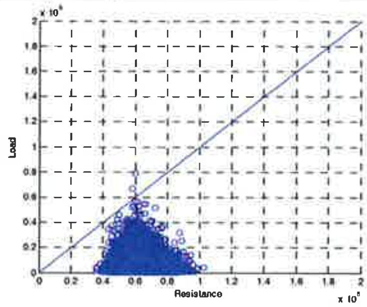

Figure 9. Result of MCS (Confined, El Centro, NSCP 2010): Pf=0.000020, β=4.107480

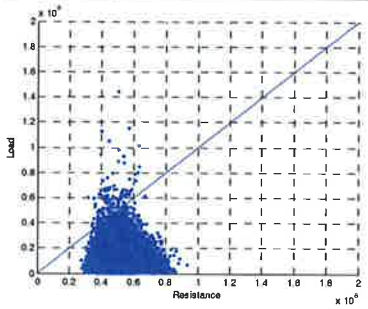

Figure 10. Result of MCS (Unconfined, Tohoku-Kanto, NSCP 2001): Pf=0.001460, β=2.976037

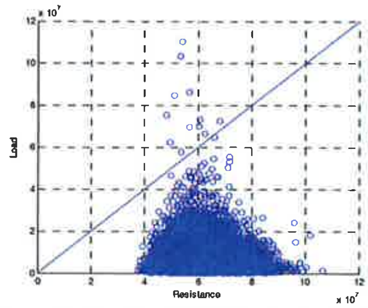

Figure 11. Result of MCS (Confined, Tohoku-Kanto, NSCP 2001): Pf=0.000120, β=3.672701

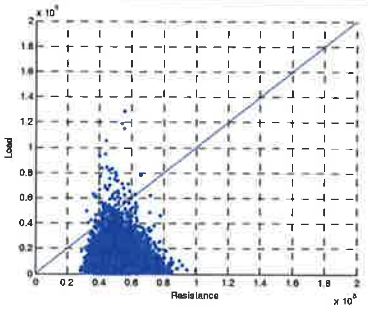

Figure 12. Result of MCS (Unconfined, Tohoku-Kanto, NSCP 2010): Pf=0.001090, β=3.064547

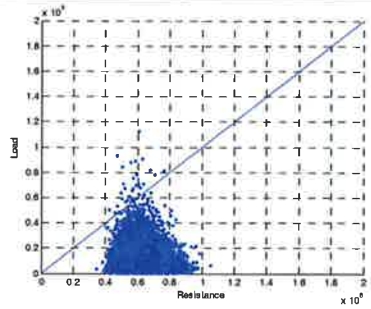

Figure 13. Result of MCS (Confined, Tohoku-Kanto, NSCP 2010): Pf=0.000390, β=3.359796

Analysis

Referring to Table 4, based from NSCP 2010 for the minimum design load combination, there is a significant decrease of probability failure, from 0.109% to 0.039%, with a per cent difference of 64%, when the confinement to the pier was incorporated while subjecting it toTohoku-Kanto Earthquake simulation. A similar scenario can be observed that of El Cenro Earthquake simulation, but with a higher per cent difference of 91% in the probability of failure decrease.

Comparing this to NSCP 2001, there is a significant decrease of probability of failure, from 0.146% to 0.012%, with a per cent difference of 92%, when the confinement to the pier was incorporated while subjecting it to Tohoku-Kanto Earthquake simulation. A similar scenario can be observed that of El Centro Earthquake simulation, but with a lower per cent difference of 55% in the probability of failure decrease.

Comparing the probability of failure of RC column for the Tohoku-Kanto Earthquake simulation, there is a decrease in per cent difference of 28% from NSCP 2001 to NSCP 2010 minimum design load combinations. But in the case of the El Centro Earthqukae simulation, there is an increase in per cent difference of 36%.

Comparing the probability of failure of unconfined RC column (to both simulated earthquakes) for the load combinations with respect to NSCP, there is maximum of 0.00037 (the other is 0.00006) decrease of P_f from that of NSCP 2001 to NSCP 2010. In the case of the confined RC column, there is a maximum of 0.00027 increase of P_f from that of NSCP 2001 to NSCP 2010, after simulating it to Tohoku-Kanto earthquake. But for the case of El Centro earthquake simulation, there is a decrease of 0.00011 P_f.

Discussion, Conclusion, and Recommendation

The reliability indices of the light railway transit, specifically in one of its reinforced concrete pier, is 3.06 (unconfined, NSCP 2010) and 3.67 (confined, NSCP 2001) when it was simulated under a Tohoku-Kanto Earthquake. A similar scenario was also computed for the simulation of El Centro Earthquake, that is, 3.50 (unconfined, NSCP 2010) and 4.10 (confined, NSCP 2010). This can be attributed to the effectiveness of the confinement model used in this simulation, that is, a maximum of 92% improvement of confinement in the reinforced concrete pier.Based from the ordinary Monte Carlo Simulation, the light railway transit, in its maiden structural form, can withstand seismic forces, given that a confinement model must be chosen for the design of the reinforced concrete pier. A decrease of P_f from NSCP 2001 to NSCP 2010 can be attributed to a more conservative load combination for a Level 2 simulated earthquake, but with a Level 1 simulated earthquake, NSCP 2001 load combination is more conservative than the current local code.

To further strengthen this claim, the researcher proposes the following for future findings:

- Consider other failure modes, e.g. foundation uplift, shear failure.
- Update the strength of the structure using data from Non-destructive Test. In turn, these data would be used in a method of structural reliability to obtain the value of reliability index or probability of failure of the structure. Since this research dealt with a stead state condition, it is suggested to refer to[18], [19], and [20].

- A series of known ground motion data, specifically ground acceleration taking into account the soil type, must be used to compute for the reliability index. See [8].
- Instead of a simple SDOF lumped mass model, structural modeling thru the use of finite element methods must be used for an accurate account of the physical properties of one of the LRT's reinforced concrete pier. Numerous commercially available software package can be used for the implementation of FEM, but still MatLab is a powerful tool for a seasoned structured programming trained researcher. See[17], [11] and [18].
- Since this research dealt with a structural component reliability, a systemreliability study can be implemented using the as-built plans of a certain line of the LRT System.

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