Statistical Analysis of the Damage of Bridges during Earthquakes

Abstract

Bridges are vital links in the transportation systems and are specially important for effective post-earthquake recovery operations. The current study is geared towards statistical analysis to quantify the extent of damage due to earthquakes on box girder bridges in California which were constructed before 1971. It is found that the mean value of the damage, irrespective of single or double column bridge, falls in the moderate damage region. It is also found that the damage to the bridges correlates positively with the maximum peak ground acceleration during an earthquake, and does not correlate with any of the structural or geometrical properties of the bridges.

Contents

1. Introduction

Bridges are considered to be the lifeline of modern world because of their importance in the case of natural disasters such as earthquake, cyclones etc. It is necessary that the bridges should be in functional state after such disasters as it facilitates effective post recovery operations. A quantity that has been used to quantify the severity of the ground motion occurred during an earthquake is the peak ground acceleration (PGA, S_a). Past earthquakes (San Fernado (1971), Kobe (1995), Loma Prieta (1998), etc) have shown that bridges can suffer significant damage during earthquakes.

Bridges are considered structurally deficient if significant load-carrying elements are found to be in a poor or worse condition after an earthquake. A majority of bridge seismic failures in the past are attributed to column failures. Columns supporting bridge structures may be expected to respond inelastically during strong earthquakes. The nature of failure for columns may depend on a number of factors such as cross sectional area of the columns, longitudinal and transverse reinforcement ratios, column height, concrete compressive strength and reinforcing steel yield strength, axial load on the column etc. California has close to 29,000 bridges which vary in age based on their time of construction. Ramanathan (2013) carried out finite element

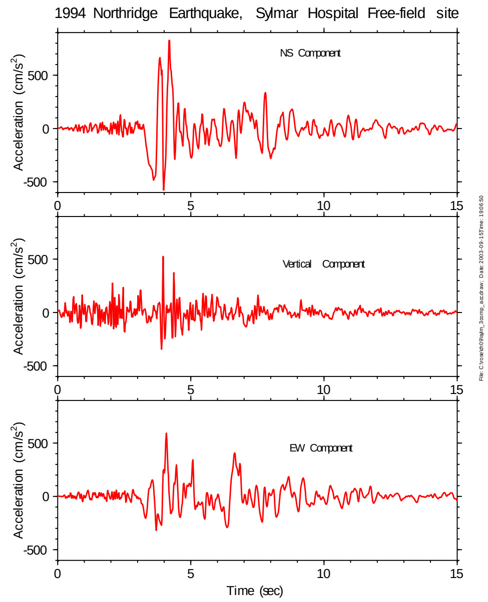

Figure 1: Peak ground acceleration (Courtesy, USGS)

analysis of bridges at California to study the response of bridges for various earthquakes. He statistically described the major geometric and strength parameters of bridges based on previous research (NBI (2010)) and field observations. The major geometric parameters were the height of bridges (H_c), the area of main bars (A_{st}). The major strength parameters consisted of the characteristic strength of concrete (f_c) and the yield strength of steel (f_y). He also considered the uncertainties in mass through a parameter called mass factor (m).

Data for one particular bridge class (Single frame box girder bridge designed before 1971), were used in the current study. The data consists of response, for various earthquake ground motions, of two type of box girder bridge: single column box girder Bridge (referred hereafter as single column bridge) and box girder bridges with two columns (referred hereafter as double column bridge) . The data was used to quantify the extent of damage as a result of a current suite of ground motions. Analysis of Variance (ANOVA) tests were carried out to check which bridges (single column or double column ones) are better in enduring these ground motions. Finally linear regression analyses were carried out to fit predictive models to measure the damage as a function of S_a, H_c, A_{st}, f_c, f_y and m.

A brief explanation of the data used in the current study is given in the following section.

2. Data

Ramanathan (2013) identified the most common bridges in California and conducted a detailed analysis to statistically describe the major geometric parameters. Data were gathered over the three significant eras separated by the historic San Fernado (1971) and Loma Prieta (1998) earthquakes. He also generated non-linear finite element models of the chosen bridge classes and carried out finite element analysis of bridges to study the response of bridges for various earthquakes. He used the suite of 140 ground motions assembled by Baker et al. (2005) for the Pacific Earthquake Engineering Research (PEER) transportation research program and these ground motions accurately represent the seismic hazard at California. A quantity that has been used to quantify the severity of the ground motion occurred during an earthquake is the peak ground acceleration (Figure 1) and this is used in the current study to characterize the ground motion. Readers are advised to refer to Ramanathan (2013) for a more detailed description of the finite element modeling, bridge parameters, ground motion suites etc.

As pointed out earlier, the current data comprise two types of bridges- single column bridges (Column diameter of 72") and double column bridges (column diameter of 48"). A typical layout of the bridge is

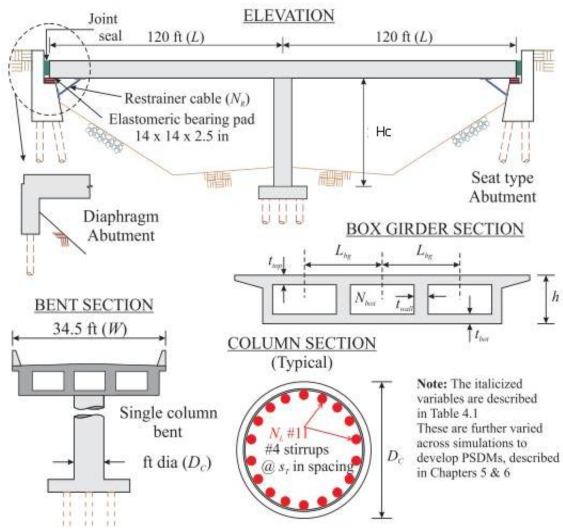

Figure 2: Typical layout of a bridge used for finite element studies

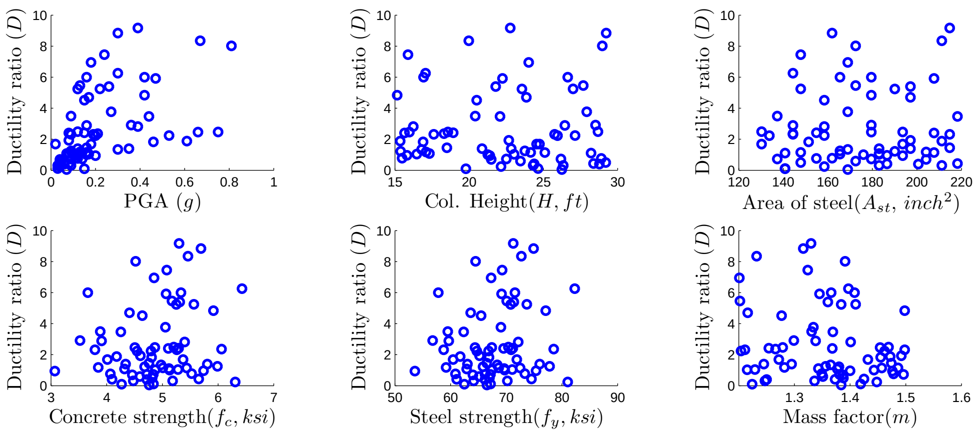

Figure 3: Scatter plot for single column bridge.

shown in Figure 2 with all the geometric parameters. The data set consists of geometrical parameters (H_c, A_{st}), Strength parameters (f_c, f_y), mass factor (m) and peak ground accelerations (S_a). Although there are many ways to measure the damage of bridges during earthquakes, curvature ductility (μ_c) is considered as the damage measure in the current study. Ductility may be defined as the ability to undergo deformations without a substantial reduction in the flexural capacity of the member (Park and Ruitong (1988)). The curvature ductility can be defined as the ratio of ultimate curvature (ϕ_{cu}) to the yield curvature (ϕ_y). The ultimate curvature is when the concrete compression strain reaches a specified limiting value and the yield curvature is when the tension reinforcement first reaches the yield strength member (Park and Ruitong (1988)).

Scatter plots of the data for the single column bridges are shown in Figure 3. The ductility ratio, which is the dependant variable in the current study, is plotted along the y-axis and the independent variables (S_a, H_c, A_st, f_c, f_y and m) are plotted along the x-axis. From Figure 4, a clear positive correlation can be seen between the variables S_a and μ_c. The correlation between other variables and μ_c is not clear from the scatter plot and will be studied in detail in the linear regression analysis (Section 6). Similarly, Figure 4 shows the scatter plots of the data for the double column bridges.

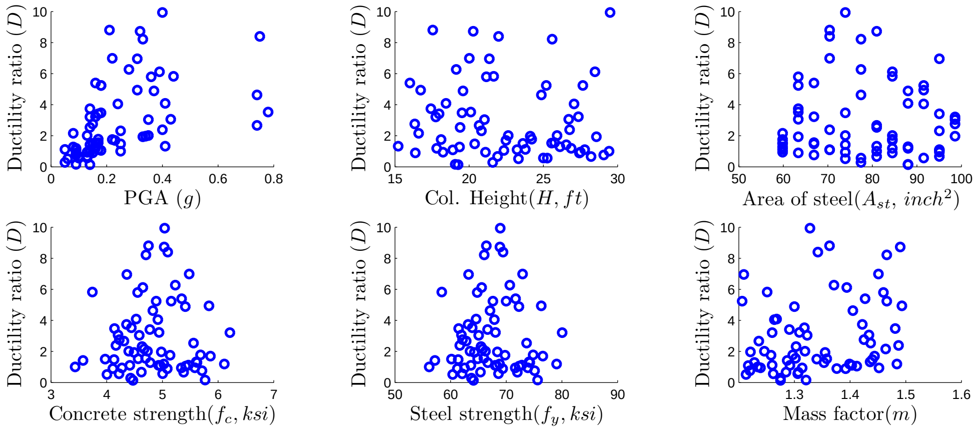

Figure 4: Scatter plot for double column bridge.

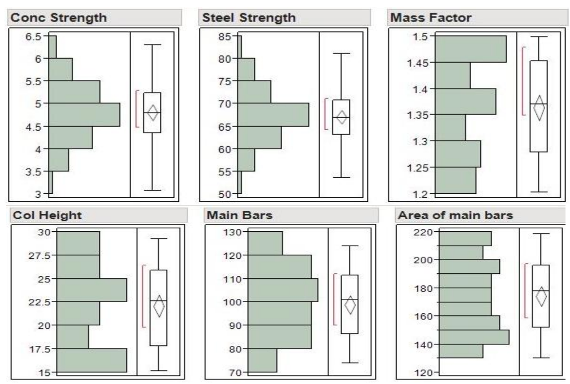

Figure 5: Histograms of the independent variables.

Histograms were plotted for each independent variable (S_a, H_c, A_{st}, f_c, f_y and m) and are shown in Figure 5. As can be seen from Figure 5 concrete strength and steel strength were sampled from a normal distribution whereas the rest of variables were sampled from an uniform distribution.

3. Relation between ductility ratio and damage

Various researchers (Nielson (2005) and Padgett (2007)) have suggested different criteria to relate the extent of damage to curvature ductility (μ_c). These criteria are shown in Table 1. For example, as per Padgett (2007), if the curvature ductility is between 1.0 and 2.0, this corresponds to slight damage. It is clear from Table 1, that the bounds of curvature ductility are different for different researchers. These bounds are derived based on extensive FEA and the data accumulated through experiments. The current data is used to classify and compare the extent of damage for the two types of bridges and is explained in the next section.

Damage states	Nielson (2005)	Padgett (2007)
Type	Curvature Ductility	Curvature Ductility
Slight damage	$1.0< \mu_c < 1.58$	$1.29< \mu_c < 2.10$
Moderate damage	$1.58< \mu_c <3.22$	$2.10< \mu_c <3.52$
Extensive damage	$3.22< \mu_c < 6.84$	$3.52< \mu_c <5.24$
Complete damage	$6.84< \mu_c$	$5.24< \mu_c$

Table 1: Relation between the extent of damage and curvature ductility. Nielson (2005) and Padgett (2007)

4. Quantifying the extent of damage of two different types of bridges

The results of our analysis for the current data are shown schematically in Figure 6. The mean ductility ratio of the double column bridges is $\bar{\mu}_{c2} = 2.92$, which is slightly higher than the mean ductility ratio of the one column bridges ($\bar{\mu}_{c1} = 2.66$). Both means, however, fall in the moderate damage region. Figure 6 also shows the corresponding 95% confident interval calculated using a t-distribution. As shown in Figure 6, for double column bridge there is overlap between the 95 % confidence interval and the extensive damage region. That indicates that they may be a significant probability that the extensive damage region includes the real mean ductility. To quantify this assertion, we calculated the probability that the extensive damage region includes the real mean ductility. We also calculated the probability that the moderate damage region includes the real mean. We repeated this analysis for both types of bridges. The probability was obtained using a t-Distribution and transforming the limits of the different regions (μ_{ci}) to their corresponding t values under our assumptions, i.e. $t_i = (\bar{\mu}_c - \mu_{ci})/(S/\sqrt{n})$. For the two column bridges we found a 15.35% chance (shown in red color in Figure 7b) that the extensive damage region includes the real mean ductility and a 84.64% chance (shown in blue color in Figure 7a) that the moderate damage region includes the real mean ductility. For the one column bridge we found a 97.28% chance that the moderate damage region includes the real mean and a 2.7% chance that the extensive damage region includes the real mean.

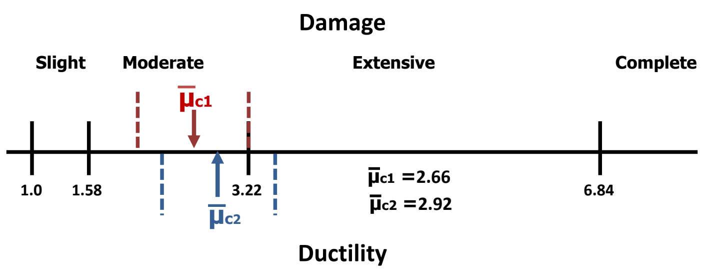

Figure 6: Mean damage and confidence intervals for the two types of bridges (The dotted red line is the 95% confidence interval for the single column bridges. The dotted blue line is the 95% confidence interval for the mean ductility of the two column bridges)

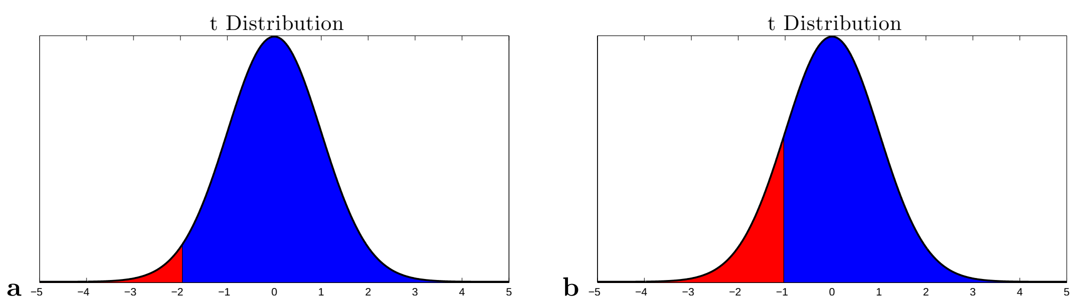

Figure 7: t-distributions for the two types of bridges. Blue corresponds to the probability that moderate damage region includes the mean ductility, red corresponds to the probability that the extensive damage region includes the mean ductility.a) Probability distribution for the single column bridge, b) Probability distribution for the double column bridge.

5. ANOVA test to compare the endurance of two types of bridges

We conducted a one-way ANOVA analysis to compare the mean ductility ratio of the two type of bridges. The null hypothesis of the test was that the mean ductility ratio for both types of bridges are the same, and the alternative hypothesis is that they are not. The P-value of the test was 0.523 with a significance level of

0.05. From this we conclude that there is not enough evidence at 0.05 significance level to state that any of the bridges suffers less damage than the other during seismic activity. However, we obtained a very low R^2 value and from the histogram and the probability plot of the residuals of the test it seems that the assumption of normality for the residuals is violated. To test this we conducted a normality test on the residuals (Figure 8) and obtained a P-value less than 0.05, indicating that there is a high probability that the residuals do not come from a normal distribution. Motivated by the shape of the histogram we transformed the response (curvature ductility) using the logarithm function, and then performed another one-way ANOVA analysis to compare the mean of the natural logarithm of the ductility for both types of bridges.

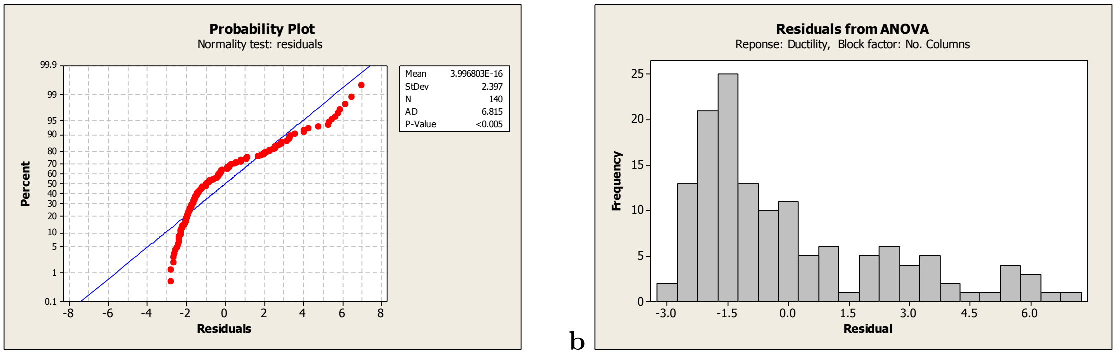

Figure 8: a) Probability plot of the residuals for the ANOVA test comparing the mean ductility for the two types of bridges. P-value suggests that the normality assumption is violated. b) Histogram of the residuals for the same ANOVA test.

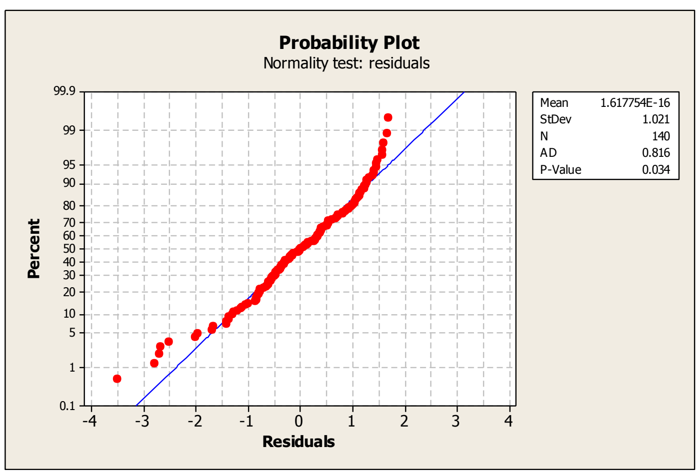

Figure 9: Probability plot of the residuals for the ANOVA test using the logarithm of the ductility.

The conclusion from this comparison is analogous to our previous conclusion. There is not enough evidence at a 0.05 significance level to say that there is a difference in the mean of the logarithms of the ductility of the two types of bridges(P-value = 0.256). In this case the P-value for the normality is 0.034, which is higher than from our previous analysis but it is still lower than 0.05 (Figure 9).

6. Multiple Linear regression analysis

As the current data contains more than one independent variable (repressor variable), multiple linear regression is carried out with the given data. The current multiple linear regression attempts to model the relation between the response variable (curvature ductility) with the explanatory variables (S_a, H_c, A_{st}, f_c, f_y and m) by fitting a linear regression to the data (Equation 1).

$$\mu_c = \beta_0 + \beta_1 S_a + \beta_2 H_c + \beta_3 A_{st} + \beta_4 f_c + \beta_5 f_y + \beta_6 m + \epsilon \tag{1}$$

The error term, ϵ (difference between the actual observation and the corresponding fitted value from the regression model) is assumed to be uncorrelated random variables, and normally distributed with zero mean and constant variance. The regression analysis is carried out with the help of commercial statistical package Minitab.

7. Multiple linear regression for single column bridge

The regression analysis obtained with the data from Minitab is given in equation 2. The standard deviation of the coefficients, T value, P-value and important parameters based on the P-value are given in Table 2. The P (probability) values for the constant coefficients measure the probability that the values are not derived by chance. These P-values are not a measure of 'goodness of fit', rather they state the confidence that one can have in the estimated values being correct, given the constraints of the regression analysis (i.e., linear with all data points having equal influence on the fitted line). A higher P value indicates that the corresponding parameter is not having much significance in the regression equation. The P-value of 0.000 is a little misleading as Minitab only calculates P-values to 3 decimal places, so this should be written as $p < 0.001$. The R^2 and adjusted R^2 values are estimates of the 'goodness of fit' of the line. They represent the % variation of the data explained by the fitted line; the closer the points to the line, the better the fit.

$$\mu_c = -8.75 + 7.16 S_a + 0.0747 H_c - 0.002 A_{st} - 3.04 f_c + 0.452 f_y - 5.26 m \tag{2}$$

The residual plot for the regression model, which contains the check for normal distribution for ϵ using the probability plot, residual versus fitted value, histogram of the residuals and the residual in the observation order is given in Figure 10. It is clear from Figure 10, that the assumption of constant variance is violated in the current regression model and needs some revision. The R^2 value for the current regression is 0.25 and the adjusted R^2 value is 0.24.

Coefficients of regression	Coefficient	Std. deviation of coefficient	T value	P value	Important / Not important
Constant	30.47	27.65	1.1	0.275	Important
S_a	6.89	1.568	4.4	0	Important
H	-0.026	0.065	-0.4	0.69	Important
A_{st}	0.004	0.022	0.18	0.854	Not Important
f_c	13.01	8.993	1.45	0.153	Important
f_y	-1.47	1.044	-1.41	0.164	Important
m	5.34	3.141	1.7	0.094	Important

Table 2: Initial regression analysis for single column bridge.

From Table 2, the variable A_{st} (which has the highest P-value) is identified as not important for a 95 % confidence level (P-value should be less than 0.05) and the regression analysis is carried out further without that variable. In this process, Minitab identifies some outlier in the given data and these were deleted from the data set. The above process is repeated (i.e. identifying and deleting variable with P-value less than 0.05 and deleting the outliers). The regression equation obtained finally is given in (Equation 3) and the importance factors in Table 3.

$$\mu_c = -0.0403 + 7.16 S_a \tag{3}$$

From Figure 11, it can be seen that the residuals follow a normal distribution and the constant variance assumption is satisfied fairly. The R^2 value for the final regression is 67.6 % and the adjusted R^2 value is 66.5 %.

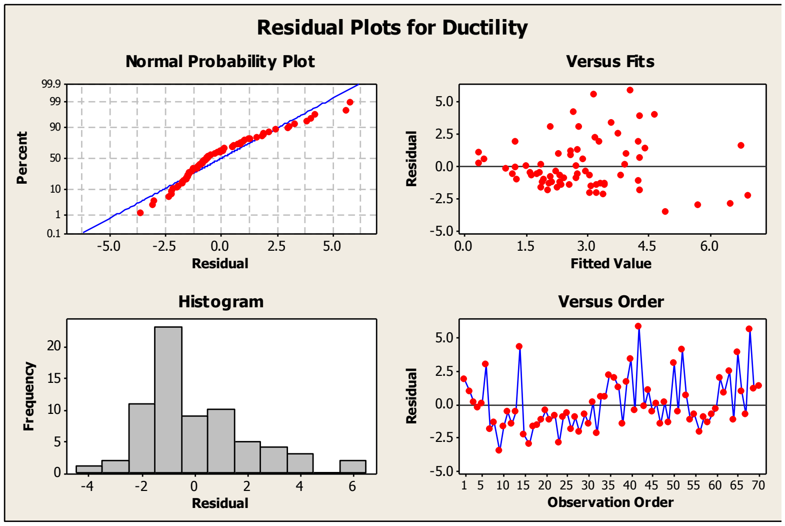

Figure 10: Residual plots for ductility for single column bridge (Initial trial).

Coefficients of regression	Coefficient	Std. deviation of coefficient	T value	P value	Important / Not important
S_a	9.367	1.185	7.9	0	Important

Table 3: Final regression analysis for single column bridge.

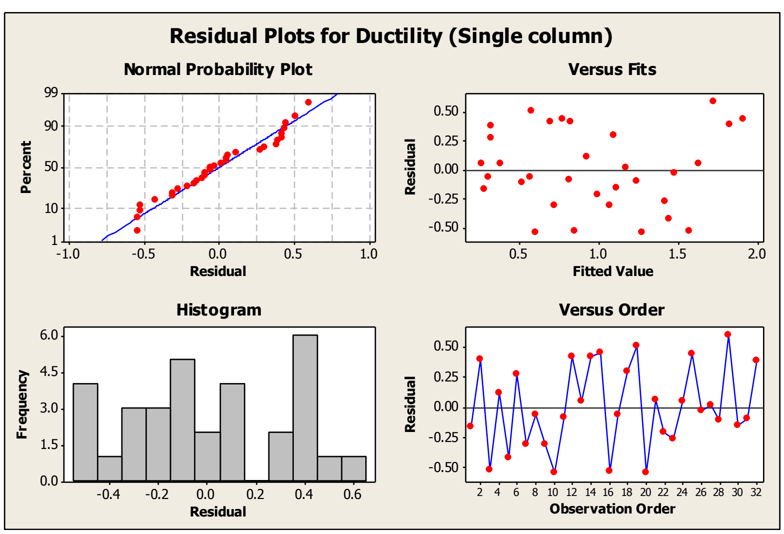

Figure 11: Residual plots for ductility for single column bridge (Final trial).

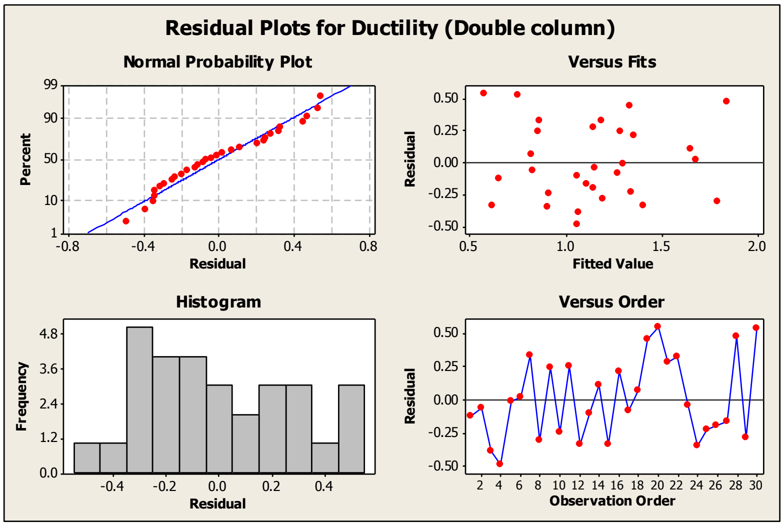

Figure 12: Residual plots for ductility for double column bridge (Final trial).

8. Multiple linear regression for double column bridge

Analogous to single column bridge, multiple linear regression is carried out for double column bridge. The procedure as explained in the single column bridge is followed in the double column bridge and the regression equations obtained by the initial trial (Equation 4) and the corresponding parameters are given in Table 4 .

$$\mu_c = 30.5 + 6.89S_a - 0.0263H_c - 0.0041A_{st} + 13f_c - 1.47f_y + 5.34m \tag{4}$$

Coefficients of regression	Coefficient	Std. deviation of coefficient	T value	P value	Important / Not important
Constant	30.47	27.65	1.1	0.275	Important
S_a	6.89	1.568	4.4	0	Important
H	-0.026	0.065	-0.4	0.69	Important
A_{st}	0.004	0.022	0.18	0.854	Not Important
f_c	13.01	8.993	1.45	0.153	Important
f_y	-1.47	1.044	-1.41	0.164	Important
m	5.34	3.141	1.7	0.094	Important

Table 4: Initial regression analysis for double column bridge.

The importance of regression coefficients are identified based on the P-value and deleted accordingly (refer to section 8) and the regression equation obtained finally is given in equation 5 and the importance factors in Table 5. The R^2 value for the final trial is 54.8% and the adjusted R^2 value is 53.2%. The residual plots are shown in Figure 8 and it can be seen that the normality and constant variance for ϵ is satisfied.

$$\mu_c = -0.0403 + 7.16S_a \tag{5}$$

Coefficients of regression	Coefficient	Std. deviation of coefficient	T value	P value	Important / Not important
S_a	6.148	1.055	5.83	0	Important

Table 5: Final regression analysis for double column bridge.

It may look surprising as the damage measure (μ_c) relates only to the peak ground acceleration (S_a), ie, the regression equation is independent of the geometric and strength parameters of the bridge.

9. Conclusion

Earthquake damage to a bridge can have severe consequences. As bridges are vital link in the transportation system, even if a temporary bridge closure has severe consequences. The current study conducted hypothesis tests to measure the extent of damage on box girder bridges due to assembled 140 suites of ground motions. It is found that the mean value of the damage, irrespective of single or double column bridge falls in the moderate damage region. For the single column bridge we found a 97.28% chance that the moderate damage region includes the real mean damage and a 2.7% chance that the extensive damage region includes the real mean damage. For the double column bridges we found a 15.35% chance that the extensive damage region includes the real mean damage and 84.64% chance that the moderate damage region includes the real mean damage. It is clear from the regression analysis that the damage depends only on the peak ground acceleration. From the given data, we did not find a correlation between structural properties of bridges and earthquake damage. It is, however, clear from experimental data that for small earthquakes a strong correlation exists between the strength, geometric parameters, and the damage measure. The data used in the current study consists of accelerations ranging from 0.02 g (small earthquakes) to 0.85 g (very large earthquakes). The reason why we are not finding such a relation can be due to the wide range of earthquakes in the current data. Also, our data set for small earthquakes is insufficient to make a statistically significant argument about the correlation between structural properties of bridges and earthquake damage. Additional data for small earthquakes could help address this question.

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