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DOCTORAL THESIS

Study on Seismic Behavior and Failure Mechanisms of Full Scale RC Bridges

March, 2015

Zhongqi SHI
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Chapter 1 Introduction

1.1 Research Background and Objectives

Being vital infrastructures of transportation facilities, bridges should have sufficient seismic capacity and safety, even under extreme condition such as earthquakes. However, in the past significant earthquakes, such as 1989 Loma Prieta Earthquake, USA, 1994 Northridge Earthquake, USA, 1995 Kobe Earthquake, Japan, 1999 Chi Chi Earthquake, China, 1999 Bolu Earthquake, Turkey, and 2008 Wenchuan Earthquake, China, a large number of bridges suffered significant damage.

During 2008 Wenchuan Earthquake, China, the RC arch bridges suffered extensive damage widely. Xiaoyudong Bridge, as shown in Figure 1.1, is one of the most famous ones, which suffered severe damage.

Figure 1.1 Damage of Xiaoyudong Bridge during 2008 Wenchuan Earthquake, China
Besides, it is widely known that the moment-axial load (M-N) interaction plays a very important role in the vibration phenomenon of RC arch bridges. On the topic of the influence on the seismic behavior of RC arch bridge by considering or neglecting the M-N interaction, several studies \[1.1 \sim 1.4\] have been performed. Though, only relatively small range of axial load variation and limited damage (around yield stage) were evaluated in these studies. Consequently, they cannot explain the failure behavior of RC arch bridge and the influence due to M-N interaction, especially under extreme axial load.

During 1995 Kobe Earthquake, Japan, a number of bridges collapsed because the RC columns which supported decks of bridges failed. A RC column damaged due to flexure at base is illustrated in Figure 1.2 as example.

![Figure 1.2 Typical Flexural Damage of RC Column during 1995 Kobe Earthquake](image)

It is known that the main cause of the flexural failure at base of RC columns is lack of flexural capacity as well as ductility capacity resulted from underestimated seismic lateral force, insufficient amount of ties and inadequate development of the lap-splice of ties. Almost all columns failed in 1995 Kobe Earthquake, Japan, were built in 1970s and designed based on the 1964 Design Specifications of Steel Road Bridges \[1.5\]. A number of bridges built in 1970s were retrofitted after 1995 Kobe Earthquake, Japan. However, the bridges, which have similar properties with the column failed in 1995 Kobe Earthquake, Japan, and are not retrofitted, still exist in other area where strong earthquake may hit on. Therefore, it is of great importance to
investigate the failure mechanisms of these types of columns failure in 1995 Kobe Earthquake.

On the other hand, it was reported \cite{1.6} that the RC columns also suffered significant residual displacement although the apparent damage might be not notable in 1995 Kobe Earthquake, causing the difficulty or impossibility to retrofit. About 100 columns with a tilt angle of more than 1° (1.75% drift) were demolished and new columns were built \cite{1.7}, due to the difficulty that setting the superstructures back to the original alignments and levels would have involved.

Then, in 2002 Specification \cite{1.8}, it was defined that the residual displacement should not exceed 1% of the height of pier to ensure the safety. However, only few studies were published focusing on the residual displacement of RC column of bridge. For study on actual damage of RC column in 1995 Kobe Earthquake \cite{1.6}, general damage condition was summarized, and it was proved based on numerical analyses that the E-W component of Takatori wave might induce only small hysteresis residual response. Besides, other studies (example as Ref \cite{1.9}) based on experimental tests, only discussed the hysteresis residual response of single RC column. Thus, the influence from neighboring structure, such as that from the frictional resistance on side bents, has not been studied by comparing experimental result and analytical result.

Despite the variations of structural properties and failure characteristics, it is possible for researchers to gain insight into structural behavior and to identify potential weakness in existing and new bridges, by studying the failure mechanisms of actual bridges or based on experimental tests. On one hand, bridges failed in actual earthquakes have provided valuable real examples to be studied. On the other hand, experimental tests have been widely performed to provide more measurable and observable damage progress. However, noticeable influences due to scale effect and loading protocol cause notable differences between experimental result and reality.

Therefore, focusing on the seismic behavior and the failure mechanisms of RC bridges with low ductility, the assessment of actual bridge structure, which was affected in actual earthquake, and the evaluation on the full scale experimental specimen by excitation table tests, were presented in this study on following topics:

**Topic 1. The failure mechanisms of RC rigid-frame arch bridge;**

**Topic 2. The influence by axial load variation on the seismic behavior of RC rigid-frame arch bridge and its failure;**

**Topic 3. The mechanisms of noticeable residual displacement occurred to the RC column;**

**Topic 4. The mechanisms of shear failure for RC column with cut-off of longitudinal bars.**

Therefore, based on the P-δ relationship, the connection between these four topics are illustrated in **Figure 1.3.**
In this figure, the P-δ relationship for normal reinforced concrete column, with normal reinforcement arrangement and under normal range of axial compression, is plotted in black solid line. Compared to this curve, the objective bridge, a full scale reinforced concrete arch bridge collapsed in 2008 Wenchuan Earthquake, in Topic 1 and Topic 2 suffers comparatively high axial compression, which results in the decrease of resistance for not only the yield stage, but also the ultimate stage. This phenomenon will be discussed in details in Chapter 3 (Topic 1), followed the discussion on the influence due to axial load variation presented in Chapter 4 (Topic 2).

On the other hand, the attention of Topic 3 is mainly paid on the residual response behavior of a normal concrete column, with normal reinforcement arrangement and under normal range of axial compression. Therefore, this topic focuses on the behavior of the full scale reinforced concrete column after reaching its ultimate stage. At last, Topic 4 focuses on the mechanisms of shear failure for RC column with cut-off of longitudinal bars, especially on the progress of damage at the cut-off position and the degrading of shear resistance. The evaluation on these two topics, based on full scale experimental tests, is presented in Chapter 5.

1.2 Thesis Contents

Based on the background and existing problem stated above, studies are preformed and presented in this thesis.

Total six chapters are included in this thesis. The general study flow is summarized in Figure 1.4.
Chapter 1 Introduction

- **Topic 1**: Failure mechanisms of RC rigid-frame arch bridge;
- **Topic 2**: Influence by M-N interaction on RC arch bridge and its failure;
- **Topic 3**: Mechanisms of residual displacement of RC column;
- **Topic 4**: Mechanisms of shear for RC columns with cut-off.

**Research objective**: to verify the seismic behavior and the failure mechanisms of RC bridges based on full scale RC bridge, especially to clarify the influence due to axial load variation and the occurrence of residual response.

Chapter 2 Literature Review

- **2.1**: Ultimate stage and ductility of RC columns;
- **2.2**: Seismic behavior of RC arch bridges;
- **2.3**: Residual response of RC columns;

Chapter 3

**Topic 1**: General structure of rigid-frame arch bridge
- Summary of actual damage condition
- Dynamic analysis with 2-span model
- Evaluation on failure mechanisms

Chapter 4

**Topic 2**: Actual damage under high axial compression
- Evaluation on flexural-axial capacity
- Numerical study based on dynamic analyses
- Evaluation on effect due to M-N interaction

Chapter 5

**Topic 3 & Topic 4**: C1-1: flexural type RC column
- C1-1: evaluation on residual response
- C1-2: shear type RC column with cut-off
- C1-2: evaluation on shear resistance and failure

Chapter 6 Conclusions

- **Topic 1**: The exposure of pile made P3 more deformable, based on dynamic analyses. This caused more severe local failure on Span 4, especially on arch leg by axial load up to 69% of axial load capacity. Besides, local failure reduced the degree of static indeterminacy, causing the gradual loss of stability and the collapse of Span 3 and Span 4 finally;
- **Topic 2**: The flexural damage was underestimated if M-N interaction was neglected. Besides, severe failure of arch legs and collapse of Span 3 and Span 4 should have been possibly avoided by adding ties or by enlarge sectional area;
- **Topic 3**: Friction of movable bearing caused horizontal load to remain on column, leading to the notable residual displacement. The friction-free assumption by specification might not be safe, by underestimating the residual displacement;
- **Topic 4**: By early flexural crack until half of section, and following shear crack, shear angle (60°) was greater than assumption (45°), below the cut-off point. Thus, shear resistance was reduced greatly and severe shear failure occurred.

Based on the statement above, this study, focusing on the full-scale RC structures, presented the new evaluations on the failure mechanisms and on the influence due to the variation of axial load for RC arch bridges, and presented the original research on the mechanisms of residual displacement for RC bridge columns.

**Figure 1.4 General Study Flow**
Chapter 1 introduced the general research background and research topics on the failure mechanisms and the seismic behavior of full scale RC structures with low ductility under earthquake effect, as well as the research objectives.

In Chapter 2, literature reviews, mainly on the ductility of RC columns, the seismic behavior of RC arch bridges and residual response of RC columns, were presented.

In Chapter 3 (corresponding to Topic 1), the failure mechanisms of a RC rigid-frame arch bridge (Xiaoyudong Bridge) were studied in details. By summarizing its actual failure in details, it was found that the exposure of pile made P3 more deformable, based on 2-span dynamic analyses. This caused more severe local failure on Span 4, especially the failure of arch leg by axial load up to 69% of axial load capacity. Besides, local failure reduced the degree of static indeterminacy, which caused the gradual loss of entire stability. Consequently, Span 3 and Span 4 collapsed finally.

In Chapter 4 (corresponding to Topic 2), the influence of M-N interaction on arch leg was evaluated based on the analyses for the collapsed Span 3 and Span 4. The flexural damage was found underestimated if M-N interaction was neglected. Besides, the severe failure of arch legs, the main supporting member, and further collapse of Span 3 and Span 4 should have been possibly avoided by adding the ties volume or by enlarging the sectional area.

In Chapter 5, the seismic behaviors of a flexural type RC column (C1-1) based on full scale experiment were study (corresponding to Topic 3). Dynamic analysis was conducted, to evaluate the mechanisms, especially focusing on the occurrence of residual response. It was found that frictional force of movable bearing caused notable horizontal load to remain acting on column top. This load mainly led to the noticeable residual displacement. Besides, the friction-free assumption according to specification might not be safe, since it might lead to the significant underestimation of residual displacement. On the other hand, the failure mechanisms of a shear type RC column (C1-2) with cut-off longitudinal bars based on full scale experiment was studied (corresponding to Topic 4), especially on the progress of damage focus on the cut-off position and the degrading of shear resistance in details. It was found that due to early horizontal crack until half of section by flexure, and following diagonal crack by shear, the actual shear angle (60°) is greater (than assumed 45° in specification), below the just point of upper cut-off point. Thus, with smaller concrete area and less hoops to resist shear load, the total shear resistance was reduced greatly and severe shear failure occurred to this height.

In Chapter 6, the conclusions drawn according to this study were summarized.

References

[1.1] Inokuma. Y. and Daihara, N., “Seismic Design of a Steel-Concrete Composite Arch


Chapter 2 Literature Review

2.1 Ultimate Stage and Ductility of RC Columns

2.1.1 Assessment of FEMA 273/274 and FEMA 356 ultimate drifts

The “NEHRP Guidelines for the Seismic Rehabilitation of Buildings” \(^{2.1 - 2.3}\) gave values of the ultimate plastic hinge rotation of RC members as acceptable limiting values for primary or secondary components of the structural system under the collapse prevention earthquake, as a function of the type, reinforcement, axial and shear force levels, and detailing of RC members. FEMA reports \(^{2.1, 2.3}\) gave values of the ultimate plastic rotation \(\theta_{pl}\) (which is approximately equal to the total minus the implied yield rotation of 0.005 rad in beams or columns, or of 0.003 rad in walls).

If the FEMA values represent a \(m-\sigma\) bound (mean \(m\) minus one standard deviation \(\sigma\) bounds), the use of total rotations \(\theta_u\) instead of plastic ones makes the FEMA values more consistent with the available data. If, on the contrary, they were meant to be average values, the use of \(\theta_{pl}\) for beams and columns offers an advantage.

2.1.2 Empirical expressions for ultimate chord rotation of RC members

The expression for the ultimate chord rotation or drift \(\theta_u\) in monotonic loading was got as following:

\[
\theta_{u,\text{mon}}(\%) = \alpha_{st,\text{mon}} \left(1 + \frac{\alpha_{sl}}{8}\right)(0.15^v)
\]  \hspace{1cm} (2.1)

where,

- \(\alpha_{st,\text{mon}}\): coefficient for the type of steel, equal to 1.25 for hot-rolled ductile steel, to 1.0 for heat treated (tempercore) steel, and to 0.5 for cold-worked steel;

- \(\alpha_{sl}\): coefficient for slip equal to 1 if there is slippage of the longitudinal bars from anchorage beyond the section of maximum moment, or to 0 if there is not;

- \(v\): \((=N/A_f\bar{c})\) axial load ratio, positive for compression.

The expression for the ultimate chord rotation or drift \(\theta_u\) in cyclic loading was got as following:

\[
\theta_{u,\text{cyc}}(\%) = \alpha_{st,\text{cyc}} \left(1 + \frac{\alpha_{sl}}{2}\right)(1 - 0.4\alpha_{wall})(0.20^v)
\]  \hspace{1cm} (2.2)
where,
\[ \alpha_{st,cyc} \]: coefficient for the type of steel equal to 1.125 for hot-rolled ductile steel, 1.0 for heat-treated (tempcore) steel, and 0.8 for cold-worked steel;
\[ \alpha_{wall} \]: coefficient equal to 1.0 for shear walls and 0 for beams or columns.

The regression was performed on all flexure-controlled tests to failure-monotonic or cyclic-giving the following:

\[ \theta_u \% = \alpha_{st} \alpha_{cyc} (1 + \frac{\alpha_{sl}}{2.3})(1 - \frac{\alpha_{wall}}{3})(0.20^\nu) \quad (2.3) \]

where,
\[ \alpha_{st} \]: coefficient for the type of steel: equal to 1.5 for hot-rolled ductile steel; 1.25 for heat-treated (tempcore) steel; and 0.8 for cold-worked steel;
\[ \alpha_{cyc} \]: coefficient equal to 1.0 for monotonic loading and to 0.6 for cyclic loading typical of load-histories applied in laboratory tests.

Therefore, by using Eq. (2.1), Eq. (2.2) and especially Eq. (2.3), authors provided us the method to show quantitatively how member deformation capacity was affected by the characteristics of the member and its reinforcement.

### 2.1.3 Ultimate curvature

Ultimate drifts or chord rotations are typically expressed quantitatively on the basis of purely flexural behavior through the concepts of plastic hinge and plastic hinge length \( L_{pl} \) in which the entire inelasticity of the shear span is considered to be lumped and uniformly distributed

\[ \theta_u = \frac{\phi_y L_s}{3} + (\phi_u - \phi_y) L_{pl} (1 - \frac{0.5L_{pl}}{L_s}) \quad (2.4) \]

Under deformation-control conditions, the plastic hinge will fail either by rupture of the tension reinforcement or when the compression zone fails and sheds its load. Depending on the confinement of the compression zone by transverse reinforcement and on other parameters, these failure modes may take place either at the full section level, or at the level of the confined core after spalling of the unconfined concrete cover. For failure of the full section prior to spalling, the corresponding ultimate curvatures are respectively

For failure due to steel rupture at elongation equal to \( \varepsilon_{su} \)

\[ \phi_{su} = \frac{\varepsilon_{su}}{(1 - k_{su})d} \quad (2.5) \]

where,
\[ k_{su} \]: compression zone depth at steel rupture;
\[ \varepsilon_{su} \]: extreme compression fiber strain when the steel rupture.
At failure of the compression zone

\[
\Phi_{cu} = \frac{\varepsilon_{cu}}{k_{cu}d}
\]

(2.6)

where,
\[k_{cu}\]: compression zone depth at steel or failure of the compression zone;
\[\varepsilon_{cu}\]: extreme compression fiber strain when the compression zone fails and sheds its load.

2.2 Seismic Behavior of RC Arch Bridges

2.2.1 Influence due to M-N interaction

It is widely known that the moment-axial load (M-N) interaction, also known as the effect by axial load fluctuation, plays a very important role in the vibration phenomenon of RC arch bridges. On the topic of the influence on the seismic behavior of RC arch bridge by considering or neglecting the M-N interaction, several studies have been performed. Evaluations based on frame model with M-\(\Phi\) relationship considering or neglecting M-N interaction are published \([2.4 \text{ - } 2.5]\). However, only the influence on the responses for the yield stage are discussed in both studies, without mentioning the effect on the ultimate stage. On the other hand, assessments \([2.6 \text{ - } 2.7]\) are also performed by comparing the case considering the M-N interaction (based on fiber model with \(\sigma-\varepsilon\) relationship input for concrete and reinforcement) with the case neglecting the M-N interaction (based on frame model with M-\(\Phi\) relationship input for sections directly). However, not only the consideration of M-N interaction, but the differences of \(\sigma-\varepsilon\) relationship and M-\(\Phi\) relationship, and the hysteresis may also affect the analytical results, which are failed to be clarified in these studies.

It should be noticed as well that in all these references \([2.4 \text{ - } 2.7]\), the fluctuation of axial load is not extremely extensive, and does not exceed 30% of axial capacity in the M-N interaction diagram.

2.2.2 Study by Liu et al. \([2.8]\)

During great earthquake, the arch lib will probably affected by extensive axial compression and bi-axial flexural moment. For the single member of the braced-rib arch bridge using concrete-filled steel tubes (CFST), the axial force normally dominates the ultimate strength, while the influence from the flexural moment is relatively limited.

The authors defined the ultimate strain of concrete based on the experimental researches by Murata et al. \([2.9]\),
\[
\varepsilon_{su} = \frac{1.474 (\frac{\sigma_y}{E_s})}{D/t/100} + 0.006
\]

(2.7)

where,
\(\sigma_y\) : yield stress of steel;
\(E_s\) : Young’s Modules of steel tubes;
\(D\) : diameter of steel tubes;
\(t\) : thickness of the wall of steel tubes.

Considering the restrict effect from the steel tube on the development of the strain of concrete, the ultimate strain calculated by Eq. (2.7) should not exceed 0.011 for normally used tube.

Besides, the definition of the ultimate stage of cross section \(M_u\) was made as the strain of the concrete at the compressive side reaches the ultimate strain as well.

On the other hand, the ultimate stage due to the compressive axial force was considered by using the equation shown as following

\[
N_{cu} = \Phi_b \kappa (0.85 \sigma_{ck} + k \sigma_r) A_c + \beta_c (\sigma_{cul} - 1.7 \sigma_c) A_s
\]

(2.8)

where,
\(\Phi_b\) : resistance coefficient of the composite member;
\(\kappa\) : reduction factor of the composite member against the overall buckling;
\(\sigma_{ck}\) : standard design strength of concrete;
\(k\) : restrict factor at the maximum stress of concrete;
\(\sigma_r\) : lateral stress at the maximum stress of concrete;
\(A_c\) : area of concrete;
\(\beta_c\) : reduction factor of the CFST due to bi-axial compression;
\(\sigma_{cul}\) : ultimate stress of steel tube;
\(\sigma_c\) : compressive stress on the steel tube due to the gravity and the concrete before hardening;
\(A_s\) : area of steel tubes.

Based on the assessment for ultimate stage of both the flexural moment and axial force, conclusion was got that CFST provided the arch rib a relative greater ultimate strength. Therefore, this merit was able to be utilized in the design of the ultimate stage assessment.

### 2.2.3 Study by Ueda et al.\(^{[2,10]}\)

Authors evaluated the influence by the redistribution of the moment on the ultimate strength of the RC arch bridge at fixed ends, and investigated the effect by the differences of the moment-curvature relationship at the base on the ultimate strength of the bridge, for a RC arch bridge with the span length of 160 m.

The ultimate moment of cross section \(M_u\) was defined as the moment when the strain of concrete
at the compressive side reaches at 0.0035.

The first ultimate stage was found at the left bottom of the arch rib. Thus, the development of plastic hinge here caused the reduction of stiffness, and the increase of the ability to rotate. Therefore, the moment at the left bottom did not continuously develop noticeably. The maximum of displacement at the ultimate stage for both the horizontal and the vertical directions were discovered occurred near the mid-span point. However, the maximum displacement at transversal direction was only 304/150,000 of the axial length, which was probably resulted in by the limited influence from the 2nd order moment.

Entirely, the bridge had greater resistance as it reached at the ultimate stage at the seismic coefficient of 0.48, which was as 2.2 times great as the design horizontal seismic intensity ($K_h=0.22$).

Since the fixed RC arch bridge was statically indeterminate with high order, the moment redistribution due to the occurrence time point, range, and the ability of rotation of plastic hinges at the bottoms of arch might significantly influence on the entire resistance and the damage mode. Furthermore, 30% to 50% increase of resistance due to moment redistribution could be found based on the non-linear analysis compared with the linear analysis.

According to the comparison between the experimental tests with 1/10 specimen and the analyses, the range of ultimate stage was confirmed to distribute near the bottom, and the modified moment-curvature relationship based on the effect of the reinforcement pull-out should be considered for the evaluation of the resistance of the arch bottom. The resistance was found about 10% to 20% greater in the experimental test than the analyses.

### 2.2.4 Study by Zhang et al. [2.11]

Authors studied the crucial effects for the structural ultimate load-carrying capacity and failure mode of existing reinforced concrete arch ribs, by the experimental investigations and the nonlinear finite-element analysis.

Two arch ribs were salvaged from an old RC arch bridge with the span length of 20 m, and then reinstalled in the lab. Experimental tests were conducted on these two arch ribs for studying the static response, the ultimate load-carrying capacity, the failure mode, and the influence of the damage conditions. Based on these tests, authors found that the vertical displacement-load relationship kept approximately linear under until about 70% of the ultimate load. Then the displacement curves drop rapidly. The load-strain curves were basically straight lines when the loading was up to 70% of the ultimate load. With further increase in loading, the nonlinearity became pronounced as well. Besides, the first sign of failure initiation was observed at the L/8 location in both the arch ribs. With the applied load approaching the failure loads, it can be
viewed that left L/8 region was obviously hoisted. Both the arch ribs suddenly broke at the west arch foot after a loud noise.

On the other hand, finite-element analysis was performed to compute the ultimate load and simulate the failure process of the arches. The predicted ultimate load by FEM for a damaged structure coincided with the experimental result well, while the ones by undamaged structure was obviously greater. The computed load-vertical displacement and load-strain relationship were approximately linear when the load was beneath 70% of the ultimate load, which also indicated the nonlinear finite-element analysis coincided with measurement well.

2.3 Residual Response of RC Columns

2.3.1 Summary of actual residual displacement of RC columns in 1995 Kobe Earthquake

According to the report [2.12], in 1995 Kobe Earthquake, Japan, bridge structures suffered destructive damage. About 250 RC columns collapsed totally due to the strong ground motion. On the other hand, About 100 columns with a tilt angle of more than 1˚ (1.75% drift) were demolished and new columns were built, because of the difficulty that setting the superstructures back to the original alignments and levels would have involved.

Detailed result based on field investigation was published [2.13], focusing on the residual displacement of RC columns in 1995 Kobe Earthquake and its evaluation. The characteristics of the residual displacement were summarized as following:

(1) In the longitudinal direction of bridges, the number of the RC columns that tilted with large residual displacement was notable at Kobe side;

(2) In the transverse direction of bridges, the number of the RC columns that tilted with large residual displacement was greater at mountain side than at sea side; besides, the values of residual displacement were more significant in the transverse direction than the other directions at mountain and Kobe side;

(3) In the east regions of Nishinomiya City and Ashiya City, the RC columns suffered relatively extensive residual displacement.

Furthermore, based on dynamic analyses, the effect of different seismic waves was discussed as well [2.13]. It was found that the Takatori wave (EW component) might have limited possibility to induce large residual displacement, while the Amagasaki wave (EW component) might trigger the most severe residual displacement, even approximately 3 times of that by Takatori wave (EW component).
2.3.2 Residual displacement response spectrum

Studies \cite{2.14, 2.15} presents the residual displacement properties developed in RC columns after earthquakes. By conducting time-history response analyses using a series of ground motions for single-degree-of-freedom systems, the influences on the residual displacement of RC columns due to the parameters of natural periods, peak response ductility factors and second stiffness ratios, were discussed. A simplified equation assessing the residual displacement in RC columns was proposed to the static seismic design.

The bilinear factor $r$ and the displacement ductility $\mu$ are defined respectively as:

$$r = \frac{k_2}{k_1}$$  \hspace{1cm} (2.9)

$$\mu = \left| \frac{u_{\text{max}}}{u_y} \right|$$  \hspace{1cm} (2.10)

where, 
- $k_2$ : second post-yielding stiffness for bilinear system;
- $k_1$ : initial static stiffness;
- $u_{\text{max}}$ : maximum displacement of the single-degree-of-freedom systems;
- $u_y$ : yield displacement.

Therefore, the maximum possible value of the residual displacement $u_{r,\text{max}}$ for a bilinear loop can be calculated as:

$$u_{r,\text{max}} = (\mu - 1)(1 - r)u_y \quad \text{for } r(\mu - 1) < 1$$  \hspace{1cm} (2.11a)

$$u_{r,\text{max}} = [(1 - r)/r]u_y \quad \text{for } r(\mu - 1) \geq 1$$  \hspace{1cm} (2.11b)

Then, the residual displacement ratio response spectrum $S_{\text{RDR}}$ can be obtained by normalizing the residual displacement $u_r$ by $u_{r,\text{max}}$ as:

$$S_{\text{RDR}} = \left| \frac{u_r}{u_{r,\text{max}}} \right|$$  \hspace{1cm} (2.12)

According to this definition, it was found that the bilinear factor $r$ affected on residual displacement ratio response spectrum $S_{\text{RDR}}$ mostly. The residual displacement ratio response spectrum is small when $r$ was positive, sharply increased as $r$ approached to zero, and is almost 1.0 when $r < -0.05$. On the other hand, the general trends in the relationship between $S_{\text{RDR}}$ and $r$ were not strongly dependent on the soil condition, the natural period, or the displacement ductility.
References


Chapter 3 Failure Mechanisms of Xiaoyudong Bridge

3.1 Damage of Bridges in Wenchuan Earthquake

3.1.1 Introduction of Wenchuan Earthquake

The Wenchuan Earthquake, also known as the Sichuan Earthquake, or the Great Sichuan Earthquake struck the Sichuan Province (31.0°E, 103.4°N, illustrated in Figure 3.1), China, at 14:28:01 CST (06:28:01 UTC) on Monday May 12th, 2008, lasting for around 2 minutes. It was a deadly earthquake that measured at 8.0 Ms by CEA (China Earthquake Administration) and 7.9 Mw by USGS (US Geographical Survey) 3.1. The earthquake's epicenter was 80 km west-northwest of Chengdu City, the capital of Sichuan, with a focal depth of 19 km. The earthquake was also felt in nearby countries and as far away as both Beijing and Shanghai-1,500 km and 1,700 km away-where office buildings swayed with the tremor.

Figure 3.1 Wenchuan Earthquake on May 12th, 2008
Collision of India with the Asian mainland during the earliest Eocene (~50 Ma) has resulted in the growth of the world’s largest organic belt, the Himalayas, and the associated Tibetan plateau. The seismotectonic evolution of China is characterized by the merger of several microcontinents throughout the entire Phanerozoic \[^{3.2}\]. The collision and associated convergence and extension has created 64 tectonic zones in China, which can be subdivided into a smaller number of tectonic “regions” \[^{3.3}\].

Thus, China is located in one of the most active seismic regions of the world that has been plagued by numerous destructive earthquakes during its long history. The most significant of the historical earthquakes, in terms of lives lost, was that which occurred in 1556. However, since 1900 China has experienced several more destructive earthquakes. The most destructive earthquake of the 20th Century were those of 1927 in Tsinghai, of 1932 in Gansu, of 1933 in Sichuan, of 1969 in Bohai Sea, of 1970 in Tonghai (Yunnan), of 1975 in Haicheng, and of 1976 in Tangshan (Hebei Province). The more recent earthquake of May 12, 2008 in Sichuan Province was the latest of the more destructive earthquakes that have struck China in the new millennium. Figure 3.2 illustrates the epicenters of historical earthquakes and seismic zones in China respectively.
Especially, Sichuan Basin developed as the result of the collision India with the Asian mainland. Thus, a great number of extensive earthquakes occurred around this region. The epicenters of 337 earthquakes with the magnitude greater than 5.0 occurred in the 10° square region near the epicenter of 2008 Wenchuan Earthquake (31.0°E, 103.4°N) from 1970 to 2011 are listed in Figure 3.3, based on the data from China Earthquake Networks Center.

From May 12th, totally 48 aftershocks with the magnitude not less than 5.0 occurred, among which the greatest reached the magnitude of 6.4 on May 25th, as shown in Figure 3.4.

Technically, the peak acceleration and velocity of Wenchuan Earthquake reached at 97.6% of $g$ [3.4] and 101.37 cm/s$^2$ [3.5] respectively. Their distribution maps are illustrated in Figure 3.5.
In the geotechnical point of view, Wenchuan Earthquake occurred as the result of motion on a northeast striking reverse fault or thrust fault on the northwestern margin of the Sichuan Basin. The epicenter and focal-mechanism are consistent with it having occurred as the result of movement on the Longmenshan fault or a tectonically related fault. The earthquake reflects tectonic stresses resulting from the convergence of crustal material slowly moving from the high Tibetan Plateau, to the west, against strong crust underlying the Sichuan Basin and southeastern China \cite{3.6}.

According to a study by the CEA, the earthquake occurred along the Longmenshan fault, a thrust structure along the border of the Indo-Australian Plate and Eurasian Plate. Seismic activities concentrated on its mid-fracture (known as Yinxiu-Beichuan fracture). The rupture lasted close to 120 sec, with the majority of energy released in the first 80 sec. Starting from
Wenchuan, the rupture propagated at an average speed of 3.1 km/s 49° toward north east, rupturing a total of about 300 km. Maximum displacement amounted to 9 m. The focus was deeper than 10 km \[^{[3.7]}\].

In a USGS study, preliminary rupture models of the earthquake indicated displacement of up to 9 m along a fault approximately 240 km long by 20 km deep \[^{[3.8]}\]. The earthquake generated deformations of the surface greater than 3 m and increased the stress (and probability of occurrence of future events) at the northeastern and southwestern ends of the fault \[^{[3.9]}\]. On May 20, USGS seismologist Tom Parsons warned that there is “high risk” of a major M>7 aftershock over the next weeks or months \[^{[3.10]}\].

Official figures (as of July 21st, 2008 12:00 in Beijing Time) state that 69,197 are confirmed dead, including 68,636 in Sichuan Province, and 374,176 injured, with 18,222 listed as missing. Approximately 15 million people lived in the affected area.

### 3.1.2 General damage condition of highway bridges in Wenchuan Earthquake

The property loss is nearly the entire 50% that caused by the damage and collapse of residential buildings and non-residential buildings including schools and hospitals, and the loss that constructed by brick masonry structure is up to 90% \[^{[3.11]}\]. The central government estimates that over 7,000 inadequately engineered schoolrooms collapsed in the earthquake. Also due to this earthquake, 19 expressways, 159 national and provincial highways and 7,605 rural highways were damaged, all of which amounts to 47,277 km, 5,560 bridges and 110 tunnels were destroyed, the total lost amount reached 65.306 billion RMB \[^{[3.12]}\].

Reconnaissance reports \[^{[3.13 ~ 3.14]}\] on the damage of bridges due to 2008 Wenchuan Earthquake, China, were published based on site investigations. A total of 33,370 km of road was reported damaged by the officials. In addition, there were 4,840 bridges and 98 tunnels that were totally or partially damaged (there are in total 18,358 bridges in Sichuan province and 1,782 highway tunnels in China) \[^{[3.14]}\]. The damage to different types of bridges, including those with simple support, arch, and continuous elements, etc., are briefly summarized for three bridges: (a) Baiha Bridge; (b) Miaoziping Bridge; (c) Gaoyuan Bridge.

(a) Baiha Bridge

Baiha Bridge is a 500 m-long viaduct with a height of 30 m. It plays an important role on the route from Dujiangyan to Wenchuan and was completed and opened to traffic in 2004. It is a reinforced-concrete (RC) continuous span bridge supported by twin column piers, with a cap beam on expansion joint and twin columns without a cap beam for the rest of the supports.

There were about five spans 100 m of bridge slab that collapsed in the location of the turning section due to the earthquake, possibly because of the narrow seating on the cap beam, the
buckling pier, and the coupling effect of bending and torsion. The pier seemed to be too brittle (not sufficiently ductile). The main rebar was not adequate and the size of the stirrups was relatively undersized. The connection between the tie beam and pier columns seemed to contain insufficient rebar as well.

Figure 3.6 Damage Condition of Baiha Bridge

(b) Miaoziping Bridge

Miaoziping Bridge is located at the water reservoir area (Zipingpu dam) near Dujiangyan. This beautiful bridge is 1436 m long with a height over 108 m composed of the main bridge (long span continuous box-girder bridge) and 19 approach spans (T-girder simple supported bridge). The construction of this bridge was completed but had not yet been opened to traffic.

Figure 3.7 Damage Condition of Miaoziping Bridge
(c) Gaoyuan Bridge

Gaoyuan Bridge was a reinforced concrete bridge with simple supported girders and continuous decks.

![Damage Condition of Gaoyuan Bridge](image)

**Figure 3.8 Damage Condition of Gaoyuan Bridge**

### 3.2 Xiaoyudong Bridge

#### 3.2.1 Bridge structure

Xiaoyudong Bridge is a 189 m long, 13.6 m wide, 4 spans, rigid-frame arch bridge that was built in 1998. The overall elevation figure of the bridge viewing from the upstream side is shown in **Figure 3.9**.

![Overall Elevation Figure of Xiaoyudong Bridge](image)

**Figure 3.9 Overall Elevation Figure of Xiaoyudong Bridge (view from upstream before the earthquake, unit: mm)**

Due to the lack of design drawings of Xiaoyudong Bridge, the detailed dimensions and the reinforcement information have been assumed based on the results of field survey by using measuring tape and total station, and referred from another rigid-frame arch bridge (Jinzhai No.6 Bridge, Anhui, China [3.15]), which has almost the same characteristics with Xiaoyudong Bridge, as the span length, the rise, the width-girder ratio and the design seismic fortification.

China\textsuperscript{[3,18]} are used as guidelines. Thus, detailed condition of reinforcement, including the arrangement, numbers and diameters, has been assumed.

Therefore, based on the field investigation and the assumption, dimensions of half span and of piers are illustrated in Figure 3.10 and Figure 3.11 respectively.

\textbf{Figure 3.10 Elevation of Half Span (unit: mm)}

\textbf{Figure 3.11 Dimensions of Piers (unit: mm)}

We can see that, the arch leg (Point A in Figure 3.10) and the inclined leg (Point B) has about $21^\circ$ and $40^\circ$ slope respectively. The arch frame is formed by one arch leg from left pile cap, the corresponding one from the right cap, and the girder in the middle span. This arch frame composes one single rigid-frame, together with two inclined legs, and the girders (Point C) at the ends of deck. One span consists of five rigid-frames connected by several crossing beams (Point D in Figure 3.10), arch slabs (Point A in Figure 3.11), and extending slabs (Point B in
Figure 3.11). Spans were connected by piers and abutments to form the entire bridge.

A pier consists of a reinforced concrete moment resisting frame with two columns and a beam, upon which two decks were simply supported. The inclined legs and the arch legs from two decks next to each other were connected to a pile cap which was supported by reinforced concrete piles. There are two piles under Pier 1 and Pier 3 for each, and four piles under Pier 2, which causes that the capacity of Pier 2 is significantly greater than that of Pier 1 and Pier 3. Besides, because there was soil covering the bottom of abutments, whether there are piles under the abutments or not is still unable to know.

The reinforcement conditions of some important cross sections are illustrated in Figure 3.12, the positions of which are marked in Figure 3.10.

![Figure 3.12 Reinforcement Condition of Important Cross Sections (unit: mm)](image)

As well as the main rebars, the materials have been assumed as C30 for concrete, HRB335 for main rebars. Besides, the stirrups were assumed as HPB235 ($f_{yk} = 235 \text{ N/mm}^2$) $\Phi6.5$ mm @250 mm for the inclined legs and $\Phi6.5$ mm @200 mm for the arch legs. Thus, the area ratio of stirrups is 0.076% and 0.095% relatively for the inclined legs and arch legs.

The fundamental information and the serviceability of rigid-frame arch bridge has been studied [3.19].

Rigid-frame arch bridge is a composite structure of arch bridge and inclined rigid-frame bridge. Its main configuration is made up of arch ribs and structures above the arch ribs used in the inclined rigid-frame bridge. Rigid-frame arch bridge was first built in Wuxi City as a new light type bridge in China. Compared with other type bridges with the same span and designing load standard, concrete and steel bars of rigid-frame arch bridge’s superstructure are less needed, and the desire for infrastructure and groundsill is lower. Furthermore its construction is convenient, because of less and light members. Its construction methods of are various; prefabricating building method or site casting method can be used. Many rigid-frame arch bridges have been built in China since 1980. According to an incomplete statistical observation, the accumulative total spans of rigid-frame arch bridges are more than 15 thousands kilometers.
However, along with the development of traffic, the structures are too light for today’s loads for their lower carrying capacity, deterioration has been greatly found in many of rigid-frame arch bridges.

![Figure 3.13 Typical Crack Conformation of Rigid-Frame Arch](image)

The typical crack conformation of rigid-frame arch is illustrated in **Figure 3.13**. The cracks of rigid-frame arch distribute at arch leg, inclined arch leg, chord, big node, small node and vault. Arch leg is the most important supporting member and directly determines the safety of rigid-frame arch bridge. Now destroyed damage don’t appear in arch leg, and the cracks of exceptional arch leg don’t affect carrying capacity of rigid-frame arch bridge. The cracks of chord, big node, small node and vault exist widely. The cracks of chord and vault are vertical abruption crack, and the crack width change from wide on the bottom to narrow on the top because of tension. The cracks of big node and small node are along with the direction of arch leg and inclined arch leg in node area respectively.

Besides, the deteriorations, the cracks of crossing beam, bridge deck, and wearing surface influences the serviceability of rigid-frame arch bridge noticeably.

Several reasons have been confirmed as the main factors:

i. Lower sectional dimension;

ii. Bad entirety;

iii Lower strength of bridge deck;

iv. Feebleness of joints.

Since Xiaoyudong Bridge was constructed in 1998, it might probably suffer the effects mentioned above, although qualitative enhanced design measures were given to help the design.

### 3.2.2 Actual damage

Aiming at investigating and summarizing the damage condition of Xiaoyudong Bridge as in details as possible, field surveys has been conducted for several times on August 29th ~ September 2nd, 2008, March 31st, 2009, and September 26th ~ 28th, 2009. The results of the
field survey on both the damage condition of Xiaoyudong Bridge, and the nearby surface faults are presented, as well as the recorded seismic waves.

The overall damage condition of Xiaoyudong Bridge due to Wenchuan Earthquake is shown in Figure 3.14. From this figure, we can see that Span 3 and Span 4 collapsed entirely, while Span 1 and Span 2 stood still with limited failures. Figure 3.15 is the abstracted elevation figure of Xiaoyudong Bridge. We number the spans, abutments and piers from left side as shown.

![Figure 3.14 Overall Photo of Xiaoyudong Bridge after Wenchuan Earthquake (view from upstream)](image)

![Figure 3.15 Abstracted Elevation of Xiaoyudong Bridge after Wenchuan Earthquake (view from upstream)](image)

The detailed damage condition will be explained by each part of the bridge. Figure 3.16 illustrates the detailed damage condition of Span 1 and A1. We can see that Span 1 moved about 75 cm downwards at middle span (Point A) and the girder collided into A1 (Point B) about 90 cm, which consequently caused the shear failure of side wall (Point C). Because the settlement of A1, the arch legs collided with the revetment next to A1 (Point D), great shear failures occurred to the bottom of arch legs (Point E) and the top of inclined legs (Point F). Besides, some cracks occurred to the bottom of the legs (Point G) on Pier 1. The detailed damage condition will be explained by each part of the bridge.
For Span 2, the damage is relatively slight that the middle span moved about 10cm upwards and some cracks have been observed at the bottoms of both inclined legs and arch legs, as shown in Figure 3.17.

As shown in Figure 3.18, Pier 3 (at Point A) tilted averagely 7.5° toward A2 (about 8.08° at the upstream side and 6.85° at the downstream side of the bridge, measured by the electronic total station). The piles under Pier 3 (at Point B), beneath the retrofitting of the steel jacket, suffered great damage because of the tilt. Span 3 and Span 4 collapsed entirely, and the legs on them failed as well.
It should be noticed that at the joints of girder and arch legs, different types of failure occurred to left and right. On the left (view from downstream, Figure 3.19), by the negative moment, failure occurred because the reinforcement on the upside of girder resisted tension while the downside concrete resisted compression. Differently on the other hand, the girder on the right (Figure 3.19) was pulled by positive moment to separate from the joint, which caused extensive cracks at the joint.

Further, as shown in Figure 3.20, there are shear failure on the side wall of A2 (Point A in Figure 3.20), and a permanent displacement of the support about 20cm in the backsoil side (Point B) due to the collision between the deck of Span 4 and A2. Span 4 dropt from support, and the legs also suffered failures.
As illustrated in Figure 3.21, it has been found that the length of every span becomes 41.203 m and 42.298 m for the left two spans respectively, and 84.448 m for the right two spans totally (by noticing the tilt of P3, the right two spans are judged together), based on the result of electronic total station as well.

Considering the length of support (set as 400 mm, as 1/2 of the width of the beam upon the pier), the length of them becomes 42.003 m, 43.098 m, and 85.248 m respectively. By compared these lengths by total station with the lengths by measuring tape, (taking Span 2 as an example, use 43.098 m by total station to minus 43.150 m by measuring tape, shortening of 0.052 m can be got) the changes of span length have been got. Span 1 has an assignable shortening of 35 cm (0.82% decrease), which is probably caused by the surface faults. Span 2 has a shortening of 5 cm (about 0.12%).

On the other hand, for the right two spans, the total length decreased 25 cm (about 0.29%). Thus, the changes of span length from Span 2 to Span 4 are relatively small and therefore are able to be ignored.
3.2.3 Nearby surface faults and recorded seismic waves

As being mentioned in Sub-section 3.1.1, Beichuan-Yingxiu Fault and Guanxian-Jiangyou Fault are the main two faults due to Wenchuan Earthquake (as respectively Fault-1 and Fault-2 drawn in Figure 3.22). Another fault, Fault-3, named Xiaoyudong Fault connected these main two. Our objective bridge, Xiaoyudong Bridge was located between that two main surface faults and on this connecting Xiaoyudong Fault.

![Figure 3.22 Surface Faults Near Xiaoyudong Bridge and Seismic Stations](image)

Besides, according to Ma et al [3.23] and Kawashima et al [3.1], a surface fault went through the left dyke at about 70 m upstream (Point A in Figure 3.23). This fault displacement extended to downstream along the left dyke and crossed the road at 10 m (Point B) and 50 m (Point C) behind A1. On the other hand, no obvious trace of surface fault has been found at the right dyke.
Also mentioned in Sub-section 3.2.2, the change of span length of Span 3 and Span 4 (shortened by 25 cm totally, 0.29%) was relatively lighter than that of Span 1. Thus, it can be inferred that, the surface fault mainly effected on Span 1 (shortened by 35 cm, 0.82%).

To choose the proper seismic wave for the following dynamic analysis, the locations of three seismic observing stations were illustrated in Figure 3.22 as well. The wave forms are illustrated in Figure 3.24 and Figure 3.25 respectively for Station A (Bajiao Station) and Station C (Wolong Station). The response acceleration spectra are shown in Figure 3.26.
Firstly, since Xiaoyudong Bridge is a RC arch bridge, the U-D component of the seismic wave is of great importance for the dynamic analysis, Station B (Qingping Station) is not used for the lack of the data in the U-D component. On the other hand, Station C (Wolong Station) was isolated from Xiaoyudong Bridge by two layers of surface fault. Thus, the wave data by Station A (Bajiao Station) is utilized as the input wave for the standard case of the dynamic analysis.

**Figure 3.25 Wave Form of Wolong Station**

**Figure 3.26 Response Acceleration Spectra**
3.3 Analytical Modeling and Condition

3.3.1 Analytical modeling

As shown in Figure 3.27, on the right angle direction of the axis of the bridge, noticing five arch frames which have been arranged together to form one span, here select one single arch frame, including micro-bending slab, to establish the model. Since there are only two columns for each pier, the properties of the column have been multiplied by 2/5 to fit the single frame.

Table 3.1 listed the moment of inertia for different sections. Here, the y-axis is set as the transverse axis, while the x-axis is set as the longitudinal axis of the bridge. Therefore, the moment of inertia around y-axis and x-axis respectively corresponds to the movement in the longitudinal direction (LG) and the transverse direction (TR). The calculation of moment of inertia is conducted separately for legs, girder, and piles for P1/P3 or P2. The ratio between the moment of inertia for TR to LG is also compared. It can be observed that the movement in the TR direction is significantly greater than the movement in the LG direction. The ratio for legs, girder, and piles of P1/P3 is respectively 137, 376 and 140 times. This suggests that the movement in LG direction is probably the dominant movement, compared with the movement in TR direction of this bridge. As a consequence, it is suitable to establish merely a 2-D model to study the behavior of this bridge.

![Figure 3.27 Cross Section of Deck and Girder, and the Single Frame Model](image)

<table>
<thead>
<tr>
<th>Moment of Inertia (unit: m^4)</th>
<th>Legs</th>
<th>Girder</th>
<th>Piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>I_y LG</td>
<td>2.71×10^{-1}</td>
<td>2.37×10^{-1}</td>
<td>1.03×10^{0}</td>
</tr>
<tr>
<td>I_x TR</td>
<td>3.71×10^{3}</td>
<td>8.91×10^{1}</td>
<td>1.44×10^{2}</td>
</tr>
<tr>
<td>Ratio (TR/LG)</td>
<td>137</td>
<td>376</td>
<td>140</td>
</tr>
</tbody>
</table>

As being mentioned for the bridge structure, P2 (the pier between Span 2 and Span 3, with 4 piles beneath it) forms the symmetric axis of entire structure, and Span 3 and Span 4 collapsed entirely while Span 1 and Span 2 stood still. Besides, the different resistance between P2 and P1/P3 can be confirmed as well according to Table 3.1. Consequently, Span 1 and Span 2 and
Span 3 and Span 4 can be divided naturally according to not only the structural characteristics but also their actual damage condition. Therefore, 2-span models are established for Span 1 and Span 2 and Span 3 and Span 4 separately to verify their possible failure mechanisms.

As shown in Figure 3.28, among five arch frames on transversal direction, one single arch frame is selected, including slab, to establish 2D model. The pile beneath P3 has been exposed before Wenchuan Earthquake probably due to scouring, which is a special character of Span 3 and Span 4 distinguishing from Span 1 and Span 2 (Figure 3.29). After this event, about 7.5° residual tilt, and great damage were observed for P3 and at bottom of pile near the ground surface respectively. Thus, aiming at taking the damage of pile beneath P3 into account, tri-linear M-Φ model is set for it, to simulate the exposure of it before the occurring of earthquake. Rigid elements have been set to the following parts: the footing, the beam on the top of the piers and the joints between legs and girder. Tri-linear M-Φ elements calculated based on Japanese specification [3.21 - 3.22] are used for girder and inclined legs, considering axial forces when only dead load acts on the structure.

![Figure 3.28 Analytical Model for Span 3 and Span 4](image1)

![Figure 3.29 Difference between Span 1 and Span 2, Span 3 and Span 4](image2)

**3.3.2 Analytical conditions**

Furthermore, special attention is paid on arch legs, whose response axial load was found
significant in pre-analysis. Response range of flexural moment-axial load (M-N) at right bottom of arch leg on Span 4 (AL-4-R) is shown in **Figure 3.30**. Here, axial resistance subjected to only compression is defined as maximum axial compressive load \( N_{\text{max}} = bd*f_{ck} \).

It can be got that, when only dead load acts on the bridge (noted as Point A) the axial load on arch leg is 1396 kN (30.3% \( N_{\text{max}} \)); axial load at peak of ultimate moment (\( M_{\text{peak}} \), noted as Point B) is 1850 kN (40.1% \( N_{\text{max}} \)); and the maximum response axial load (noted as Point C) is 2991 kN (64.8% \( N_{\text{max}} \)). This indicates that moment-axial load (M-N) interaction on arch leg may have inneglectable influence and should be taken into account. Thus, bi-linear moment-curvature (M-\( \Phi \)) relationship under variable axial load (N) is calculated. Here, Hoshikuma equation [3,23] is applied to \( \sigma-\varepsilon \) relationship of concrete, thanks to its good applicability to members with low tie ratio (\( \rho_t \) is only 0.16% for arch leg). Then, the calculated \( M_y-N \) and \( M_u-N \) are illustrated in **Figure 3.30**.

![Figure 3.30 M-N Interaction Diagrams and Response Range (AL-4-R)](image)

Correspondingly, the M-\( \Phi \) relationship under three axial load conditions and N-\( \Phi \) interaction curves are shown in **Figure 3.31** (a) and (b) respectively. It can be observed that, from Point A to Point B, resistance moment increases slightly due to greater axial load, while the ultimate curvature (\( \Phi_u \)) drops from 0.00860 1/m to 0.00688 1/m; as axial load increases after the peak point until Point C, moment resistance begins to decrease, while the ultimate curvature (\( \Phi_u \)) drops further from 0.00688 1/m to 0.00467 1/m. This ultimate curvature (\( \Phi_u \)) under maximum response axial load is only about half of that under only dead load.
For the boundary conditions, vertical, horizontal and rotational springs are set under piers and abutments. For springs between girder and pier, a frictional spring which is assumed to be comparatively weak, and a supporting spring are used on pier. On the other hand, frictional and supporting springs are used on top of abutment. Currently, collision spring is not taken into account.

The response acceleration spectra, as shown in Figure 3.26 formerly, are used to make comparison of the seismic wave by Bajiao Station and Wolong Station (E-W component is compared here because the direction of Xiaoyudong Bridge is approximately along the East-West direction, while U-D component is used because of its importance for arch bridge). We can see that Bajiao wave can represent the seismic characteristics of Wenchuan Earthquake for its strong E-W component in low period zone and strong U-D component in general, although it is slightly less strong than Wolong wave. Thanks to the closest distance from Xiaoyudong Bridge of 24 km, seismic wave by Bajiao Station is used in the dynamic analyses. Both E-W and U-D components of the seismic waves, as shown in Figure 3.24, are input. The peak ground acceleration of +581 gal occurred at 38.36 s for E-W direction, and -633 gal occurred at 37.40 s for U-D direction. Damping coefficient of 20% and 2% is separately utilized for the springs at basement and all other concrete elements. Rayleigh damping based on eigen-vibration analysis is applied for entire structure. The 1st and the 10th modes are used for Rayleigh damping, for their great mass ratio. Analysis starts from 0.0 s and ends at 100.0 s. For calculation, Newmark-β (β = 1/4) method is applied in numerical integration, with the time step being 1/1000 s.
3.4 Characteristics of Eigen-Vibration

3.4.1 General

Based on the analytical models established in Section 3.3, eigen-analysis is performed respectively for Span 1 and Span 2, and Span 3 and Span 4. The general result is summarized and compared Table 3.2 for the most important first 6 modes.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Span 1 and Span 2</th>
<th>Span 3 and Span 4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>sec</td>
<td>Hz</td>
</tr>
<tr>
<td>1</td>
<td>0.393</td>
<td>2.547</td>
</tr>
<tr>
<td>2</td>
<td>0.343</td>
<td>2.915</td>
</tr>
<tr>
<td>3</td>
<td>0.304</td>
<td>3.287</td>
</tr>
<tr>
<td>4</td>
<td>0.268</td>
<td>3.733</td>
</tr>
<tr>
<td>5</td>
<td>0.142</td>
<td>7.037</td>
</tr>
<tr>
<td>6</td>
<td>0.128</td>
<td>7.830</td>
</tr>
<tr>
<td>Sum.</td>
<td>92.0%</td>
<td>62.4%</td>
</tr>
</tbody>
</table>

It can be found that the natural periods of Span 1 and Span 2, and Span 3 and Span 4 are different for the 1st mode, while approximately same for the other modes. The natural period of the 1st mode of Span 1 and Span 2 (0.393 s) is about half of that of Span 3 and Span 4 (0.795 s).

Besides, the 1st mode of Span 3 and Span 4 also occupies more effective mass in the longitudinal direction, with the effective mass ratio of 17.8%, which is approximately 2.8 times of that of Span 1 and Span 2. However, the other modes dominant similar effective mass for both Span 1 and Span 2, and Span 3 and Span 4. The 1st, the 2nd, the 3rd, and the 5th modes are the dominant modes in longitudinal direction, while the 4th mode is the dominant mode in vertical direction.

3.4.2 Span 1 and Span 2

The deformation of the first 6 modes for Span 1 and Span 2 are illustrated in Figure 3.32. The deformation mainly in the horizontal direction of the 1st, the 2nd, the 3rd, and the 5th modes, and the deformation mainly in the vertical direction of the 4th mode, can be respectively confirmed.
Figure 3.32 Eigen-Vibration Modes of Span 1 and Span 2

3.4.3 Span 3 and Span 4

The deformation of the first 6 modes for Span 3 and Span 4 are illustrated in Figure 3.33. Similar to Span 1 and Span 2, the deformation mainly in the horizontal direction of the 1st, the
2nd, the 3rd, and the 5th modes, and the deformation mainly in the vertical direction of the 4th mode, can be respectively confirmed. Besides, more significant deformation of P3 at middle of Span 3 and Span 4 than P1 can be observed as well.

Figure 3.33 Eigen-Vibration Modes of Span 3 and Span 4
3.5 Results of Non-Linear Dynamic Analysis

3.5.1 Response of legs

Based on the analytical models established in Section 3.3, non-linear dynamic analysis is performed respectively for Span 1 and Span 2, and Span 3 and Span 4. Evaluation will be explained for Span 4 compared with Span 1 as representative. Flexural failure was found to domain the actual damage condition of Xiaoyudong Bridge, while shear failure was only observed for legs on A1 probably due to collision with revetment. No other shear failure was found for the other members. Therefore, the evaluation is mainly determined based on the flexural response.

As being mentioned, the exposure of pile beneath P3 was the most distinguished aspect of Span 3 and Span 4, compared with Span 1 and Span 2. The peak response of P3 and its piles is illustrated in Figure 3.34, (a) for deformation and (b) for maximum curvature distribution. It can be found that bottom of pile near the ground suffered most obvious bending with the maximum curvature being 0.0095 1/m, which is about 6.79 times of its yield curvature ($\Phi_y = 0.0014$ 1/m), but not exceeding its ultimate curvature ($\Phi_u = 0.0173$ 1/m). Therefore, maximum displacement on top of P3 reaches 6.19 cm at 41.37 s, towards abutment A2. Besides, this great flexural failure can be confirmed from the response M-\(\Phi\) hysteresis in Figure 3.35.

![Figure 3.34 Response of Pile and Pier of P3](image)
As a consequence of this severe vibration of P3, the inclined leg and the arch leg connected to it suffer more violent response, than that connected to P1. The response M-Φ hysteresis of inclined legs IL-4-L (on P3 of Span 4) and IL-1-R (on P1 of Span 1) are compared in Figure 3.36, and the response M-Φ hystereses of arch legs AL-4-L (on P3 of Span 4) and AL-1-R (on P1 of Span 1) are compared in Figure 3.37.
Max: $0.0063$ (41.78s)

Max: $0.0080$ (41.79s)

**Figure 3.37 Comparison of Response M-Φ Hystereses at Bottom of Arch Legs (AL-4-L VS AL-1-R)**

From the figure we can see that IL-4-L reaches its maximum curvature of 0.0462 1/m at 41.77 s, which is about 1.62 times of its ultimate curvature ($Φ_u = 0.0285$ 1/m). However, IL-1-R reaches only 0.0062 1/m for maximum, not exceeding its yield curvature ($Φ_y = 0.0070$ 1/m). The maximum response curvature of IL-4-L is as about 4.6 times great as that of IL-1-R. Similarly, AL-4-L reaches its maximum curvature of 0.0080 1/m at 41.79 s, which is as about 1.3 times great as that of AL-1-R. From these comparisons, it can be inferred that the more severe vibration of P3 than P1 causes more severe damage on both inclined leg and arch leg connected to P3 on Span 4 than those connected to P1 on Span 1.

To judge the possibility of whether bottom of arch leg reaches its ultimate stage, the response range of N-Φ hystereses (AL-4-L for bottom of left arch leg on Span 4 and AL-1-R for bottom of right arch leg on Span 1), and damage criteria (N-Φ_y curve for yield stage and N-Φ_u curve for ultimate stage), are illustrated in Figure 3.38.

When subjected to only dead load, axial load on arch legs ($N_{dead}$) is 1396 kN, and the ultimate curvature under this axial load ($Φ_{u-dead}$) is 0.0086 1/m. The maximum response curvature is 0.0080 1/m for AL-4-L and 0.0063 1/m for AL-1-R, respectively 93% and 73% to the ultimate curvature ($Φ_{u-dead}$), suggesting that both points do not reach the ultimate stage judging by dead load only. However, as been mentioned, axial load of arch leg varies greatly while the bridge vibrates, and the M-N interaction has inneglectable influence on the flexural response.

Furthermore, it can be found from Figure 3.38 that AL-4-L has similarly great flexural
curvature under even high axial load (around 2000 kN), while AL-1-R has only limited curvature under high axial load. Due to the decrease of ultimate curvature ($\Phi_u$) with increase of axial load, the response range of N-\(\Phi\) hysteresis of AL-4-L actually exceeds the ultimate criterion of N-\(\Phi_u\). But AL-1-R does not reach ultimate stage. It can be inferred that by considering M-N interaction and great response axial load, AL-4-L will suffer severe damage as ultimate stage, while AL-1-R will suffer only yield of longitudinal bars.

![Figure 3.38 Response N-\(\Phi\) Hystereses at Bottom of Arch Leg (AL-4-L VS AL-1-R)](image)

Then, Figure 3.39 is plotted to explain the mostly damaged position (AL-4-R). Figure 3.39 (a) shows the response curvature hysteresis compared to the ultimate curvature ($\Phi_u$) considering M-N interaction. Then, the ultimate ratio is defined as the result that the response curvature divided by the ultimate curvature varied with the response axial load at any time point as following and shown in Figure 3.39 (b).

$$\gamma = \frac{\Phi}{\Phi_u}$$  \hspace{1cm} (3.1)

where,
- $\Phi$ : response curvature;
- $\Phi_u$ : ultimate curvature varied with the response axial load at any time point.

For example the maximum ultimate ratio 2.237 is got as 41.15s, at when the response curvature ($\Phi_{41.35s}$) is 0.01197 1/m and the ultimate curvature ($\Phi_u-41.35s$) is 0.00535 1/m (this is only about 62% of that under dead load, due to increase of axial load). This maximum ultimate ratio of 2.237 suggests the bottom of right arch leg on Span 4 (AL-4-R) suffers significantly severe damage beyond ultimate stage.
### 3.5.2 Response of girder

Besides, response curvature distribution on Span 4 and Span 1 is compared in Figure 3.40, (a) for girder corresponding the elevation viewing in (b), then (c) and (d) for inclined leg and arch leg connected to P3 or P1 respectively.

From Figure 3.40 (a), we can see that Point G-4-L (left girder joint with arch leg) reaches its maximum curvature of 0.0394 1/m beyond its ultimate stage (1.40 $\Phi_u$), which is greater by about 43.8% than G-1-R (0.0274 1/m, 0.97 $\Phi_u$). On the opposite side, G-4-R (right girder joint with arch leg) reaches its maximum curvature of 0.0338 1/m beyond its ultimate stage (1.20 $\Phi_u$), which is similar to G-1-L (0.0351 1/m, 1.25 $\Phi_u$). Therefore, girder of Span 4 has generally greater response than that of Span 1. For legs, maximum response curvature at bottom of inclined leg (c1 in Figure 3.40 (c)) and bottom of arch leg (d1 in Figure 3.40 (d)) on Span 4 is 0.0462 1/m and 0.0080 1/m respectively, which is greater than that on Span 1 notably.
3.5.3 Deformation

As stated above, due to more noticeable vibration of P3 caused by pile exposure, Span 4 has generally more severe local damage at bottom of inclined legs and arch legs, and girder joints with arch legs, compared with Span 1. Therefore, the deformation of Span 4 is more violent than that of Span 1, as well. The displacement histories in horizontal direction and in vertical direction are shown in Figure 3.41 (a) and (b) respectively. In Figure 3.41 (a) the positive displacement stands for the girder moves towards abutment (Span 4 towards A2, while Span 1 towards A1), and in Figure 3.41 (b) the positive displacement means upward movement while the negative displacement suggests downward movement. From the figure we can see that girder of Span 1 has the maximum dis-placement of +1.94 cm at 41.99 s in the horizontal direction and -11.59 cm at 57.54 s in the vertical direction. On the other hand, Span 4 vibrates more violently, with the maximum displacement of +3.69 cm at 41.35 s in the horizontal direction and -15.51 cm at 58.21 s in the vertical direction, which is respectively 1.90 times and 1.33 times of that of Span 1. Besides, the more significant vibration of girder of Span 4 can be also discovered from the vibration range in response displacement histories.
To sum it up, the deformation at the time point of peak response displacement, and the damage position are shown in Figure 3.42 ((a) for maximum horizontal displacement and (b) for maximum vertical displacement). It can be found that, based on analytical result, all bottom of arch leg and inclined leg, and girder joints with arch leg on Span 4 suffered ultimate. However, only one of two girder joints with arch legs on Span 1 suffered ultimate, with the other joint and bottom of both arch legs exceeding yield stage. More severe local failures on Span 4, probably caused by greater vibration of P3, induced more significant displacement.
3.6 Evaluation on Failure Mechanisms

3.6.1 Span 1 and Span 2

Possible failure mechanisms is going to be explained based on analytical results for Span 1 and Span 2 in Figure 3.43 and Figure 3.44. Then, actual damage at typical position will be compared to analytical results in Figure 3.45.

Because it is not known whether the surface faults occurred at the beginning of the earthquake or later, the mechanisms of these two spans are illustrated by two reasons, seismic force and surface faults, separately.

As shown in Figure 3.43 (a), when the earthquake occurred, the girders moved in the longitudinal direction due to the effect of earthquake. This is one possible reason that caused the collision between A1 and the deck of Span 1. Besides, cracks occurred to the bottoms of legs on P1 and P2 probably due to the effects of earthquake.

![Figure 3.43 Failure Mechanisms of Span 1 and Span 2](image)

As illustrated in Figure 3.43 (b), the surface fault at about 10 m behind the abutment mentioned before might cause A1 suffered a movement towards the girders and a downwards settlement. This movement towards the girders resulted in the shortening of Span 1 (the shortening finally reached at 347 mm), and the collision between A1 and the deck of Span 1 (details are illustrated...
As well, extensive shear cracks developed in the side wall (Point B), while damage occurred to the end of girder and the support at the abutment (Point A) due to this collision. Thus, the enormous horizontal force from the girder caused the inclined legs suffered shear failures on the top (Point C).

At the same time, the horizontal movement and the downwards settlement of A1 together caused the arch legs collided with the revetment, which contributed mostly to the shear failures at the bottoms of arch legs (Point D), and also partly to the shear failures on the top of inclined legs (Point C).

At last, as illustrated in Figure 3.43 (c), three main damage was observed: firstly, because of the settlement of A1, and the decreases of supporting force from both the inclined legs and the arch legs, the deck of Span 1 dropt about 75cm in the middle span at last (Point E); secondly, the collision between A1 and the deck of Span 1 finally caused the girder moved about 0.9 m against the abutment; thirdly, the deck in the middle of Span 2 raised about 10cm (Point F) probably due to the seismic force and the effects from the neighborly spans, but not the surface fault.

Then, actual damage and analytical result at bottom of right arch leg on Span 2 (AL-2-R) are compared in Figure 3.45 ((a) for position, (b) for section i-i, (c) for actual damage corresponding to Figure 3.17, (d) for analytical result). For actual damage at bottom of AL-2-R (in Figure 3.45 (c)), only surface concrete at 9 cm (bottom) and 26 cm (top) from base (section i-i) crashed, while the longitudinal bars exposed but not buckled. According to analysis
(ultimate ratio distribution from bottom shown in Figure 3.45 (d)), section i-i at base of AL-2-R would suffer yield but not ultimate with ultimate ratio being 0.427.

Consequently, actual damage beyond yield but no ultimate at AL-2-R was well reappeared in the analysis.

**Figure 3.45 Actual Damage and Analytical Result (AL-2-R)**

### 3.6.2 Span 3 and Span 4

Possible failure mechanisms is going to be explained based on analytical results for Span 4 as the representative of Span 3 and Span 4 in Figure 3.46 and Figure 3.47. Then, actual damage at typical position will be compared to analytical results in Figure 3.48.

As shown in Figure 3.46 (a), due to severe damage at AL-4-R, IL-4-R and G-4-R and the opposite points on left, girder may move toward right and collide with A2 at first (corresponding to the maximum horizontal displacement towards abutment of 3.69 cm shown in Figure 3.42 (a)). The details of collision is illustrated in Figure 3.47 (a).

Due to this collision, the backward movement and the residual displacement may probably occur to A2 (corresponding to the actual damage shown in Figure 3.20). Then, with progress of local damage and further vibration, girder may move toward left and un-seat from A2 consequently, as shown in Figure 3.46 (b) and Figure 3.47 (b). As a result, girder collapses entirely into the river, and P3 tilts toward abutment. After that, Span 3 would lose the support from P3 and collapse as well.
Figure 3.46 Possible Failure Mechanisms of Span 4

Figure 3.47 Detailed Failure Mechanisms of A2
On the contrary, actual damage and analytical result at bottom of right arch leg on Span 4 (AL-4-R) are compared in Figure 3.48 (a) for position, (b) for section ii-ii, (c) for actual damage and (d) for analytical result. In actual damage, bottom of AL-4-R suffered severe damage (in Figure 3.48 (c)).

It can be observed in the figure that both core and surface concrete at 50cm from bottom (section ii-ii) crashed, while the longitudinal bars buckled but not broke off. According to analysis (ultimate ratio distribution from bottom shown in Figure 3.48 (d)), section ii-ii at bottom of AL-4-R would suffer extreme flexural damage of 2.237 times of Φ_u under high axial load.

Besides, about 18 cm (about 1/4 of sectional depth) from bottom exceeds ultimate stage in analysis. Thus, although the range that suffers ultimate stage in analysis is smaller than actual damage after collapse of girder, the capacity loss of AL-4-R was able to be reappeared in the analysis. This suggests that the analysis considering the M-N interaction simulates the actual damage at arch legs well.

Figure 3.48 Actual Damage and Analytical Result (AL-4-R)

3.7 Summary

In this chapter, the detailed damage condition of Xiaoyudong Bridge was summarized and assessed. Then, dynamic analyses for Xiaoyudong Bridge for Span 1 and Span 2, and Span 3 and Span 4 separately were performed, followed by the discussion on failure mechanisms. As a consequence, following conclusions have been drawn:

(1) According to the detailed field investigation, significant failures occurred to A1 and Span 1 probably due to the effect of the surface fault at about 10 m behind the right dyke, including about 34 cm displacement of A1 towards the middle, the collisions and failures of legs, and the drop at middle of Span 1. On the other hand, deck of Span 4 might collide
with A2. Span 3

(2) Dynamic analyses for Xiaoyudong Bridge, a typical RC rigid-frame arch bridge, were performed for Span 1 and Span 2, and Span 3 and Span 4 separately. According to the analytical result of Span 1 and Span 2, one joint on girder with arch legs of each span might suffer severe failure even beyond ultimate for (0.97 to 1.25 $\Phi_u$). However, the bottoms of arch legs and inclined legs might suffer damage beyond yield but did not exceed the ultimate stage. Thus,

(3) On the other hand, Span 3 and Span 4 vibrated more extensively due to more movable P3. The maximum response displacement at middle of Span 4 was greater than that of Span 1 by 90% and 33% respectively in horizontal and vertical direction. Besides, severe damage occurred to all girder joints with arch leg, bottom of inclined legs and arch legs, leading to the loss of entire stability. As a consequence, arch leg and inclined leg were not able to support the girder, and both spans collapsed into the river. In actual damage of AL-4-R, concrete at base 50 cm crashed, and main bars buckled but not broke off. Although damage in analysis at bottom of arch leg was slightly less severe than actual, capacity loss was well reappeared.

References


Chapter 4 Evaluation on Influence of M-N Interaction on Seismic Behavior of RC Arch Bridge

4.1 Introduction

As being mentioned in former chapters, the M-N interaction on the main supporting member, arch legs, is considered have notable influence on the structure failure, since the axial load response varied in a very large range.

This topic was discussed in some former studies. On the one hand, evaluations based on frame model with M-Φ relation considering or neglecting M-N interaction are published [3.1 ~ 3.2]. However, only influence on the responses for yield stage are discussed in both studies, without mentioning the effect on ultimate stage. On the other hand, assessments [3.3 ~ 3.4] are also performed by comparing the case considering M-N interaction (based on fiber model with σ-ε relation for concrete and reinforcement) with the case neglecting M-N interaction (based on frame model with M-Φ relation for sections directly).

However, not only the consideration of M-N interaction, but the differences of σ-ε relationship and M-Φ relationship and the hysteresis may also affect the analytical results, which are failed to be clarified in these studies. It should be noticed as well that in all these references [3.1 ~ 3.4], the fluctuation of axial load is not extensive, and does not exceed 30% of axial capacity. However, arch leg of Xiaoyudong Bridge stands 30.3% axial capacity under only dead load. Thus, they cannot explain the failure behavior of RC arch bridge and the influence due to M-N interaction, especially under extreme axial load.

Therefore, there is necessity to evaluate its influence by considering or neglecting the M-N interaction.

In former chapter, the discussion on the failure mechanisms of Span 1 and Span 2, and Span 3 and Span 4 were respectively presented according to the results of the nonlinear dynamic analysis, by taking the M-N interaction into consideration. In this chapter, the influence due to M-N interaction will be discussed by performing the nonlinear dynamic analysis with or without the consideration of the M-N interaction for the main supporting member, arch legs, by applying the collapsed Span 3 and Span 4 as representative.
4.2 Analytical Modeling and Condition

The general analytical modeling and conditions of Span 3 and Span 4 for the case, in which the M-N interaction of arch leg is considered, are same with that used in Chapter 3. On the other hand, the other case, in which the M-N interaction of arch leg is neglected, only considers the moment-curvature relationship (M-Φ) under the axial load condition induced by only dead load. The input M-N interaction, is shown in Figure 4.1, as well as the axial load condition induced by only dead load (N_{dead}).

![Figure 4.1 M-N Interaction Diagrams](image)

Besides, N-Φ interaction, and M-Φ relationship are shown in Figure 4.2 (a) and Figure 4.2 (b). In Figure 4.2 (a), the bi-linear M-Φ relationships under particular axial load conditions were compared, including that under only dead load N_{dead} = 1396 kN (30.3% N_{max}). N-Φ interaction was shown in Figure 4.2 (b) at last. By the comparison of Point A and Point C, it can be found that due to 114% increase of axial load (1396 kN to 2991 kN), the ultimate curvature decreases by about 46% (0.00860 1/cm to 0.00467 1/cm). Totally same analytical model and condition are applied for other parts, such as girder, inclined leg, pier and pile.

Furthermore, seismic wave by Bajiao Station is used in the dynamic analyses for both cases. Both E-W and U-D components of the seismic waves are input. Analysis starts from 0.0 s and ends at 100.0 s. For calculation, Newmark-β (β = 1/4) method is applied in numerical integration, with the time step being 1/1000s.
4.3 Analytical Result

Based on the analytical model explained above, nonlinear dynamic analyses are performed and the analytical results are summarized in this section. The general response will be firstly introduced in Sub-section 4.3.1, in terms of the response displacement at the middle span point on girder of Span 4 as the representative. Then, the main evaluation will be provided in Sub-section 4.3.2 concentrating on the analytical response of arch legs and their failure judgment. At last, the analytical results are summarized and presented in Sub-section 4.3.3, where the influence by considering or by neglecting the M-N interaction of the arch leg (arch rib) will be stated, and the suggestion on future design of this bridge type, the RC rigid-frame arch bridge, will be presented.

4.3.1 General result

To explain the general results in the cases considering or neglecting the M-N interaction, the response displacement at the middle span point on girder of Span 4 as the representative is illustrated in Figure 4.3. Figure 4.3 (a) shows the response displacement in the horizontal direction, while Figure 4.3 (b) shows the response displacement in the vertical direction. Here, the solid blue line stands for the analytical result by neglecting the M-N interaction. On the contrary, the dotted red line represents the analytical result by taking the M-N interaction into consideration.
For the response displacement in the horizontal direction shown in Figure 4.3 (a), it can be found that the response displacement is relatively similar. Only slight difference occurs to the response displacement after about 50 s and to the residual displacement. On the other hand, for the response displacement in the vertical direction shown in Figure 4.3 (b), it can be found that the response displacement is almost same. Therefore, it can be inferred that the influence by considering or neglecting the M-N interaction is not obvious regarding to the response displacement.

4.3.2 Response of arch leg

After introducing the influence by considering or neglecting the M-N interaction on general response, in terms of the response displacement, the influence on the response of arch leg will be discussed.

The response moment-axial load (M-N) hystereses by considering or by neglecting the M-N interaction are compared in Figure 4.4 for the left arch leg on Span 4 and in Figure 4.5 for the
right arch leg on Span 4 as representative.

Figure 4.4 Response M-N Hystereses for Left Arch Leg on Span 4

Figure 4.5 Response M-N Hystereses for Right Arch Leg on Span 4

Then, the response axial load-curvature (N-Φ) hystereres by considering or by neglecting the
M-N interaction are compared in Figure 4.6 for the left arch leg on Span 4 and in Figure 4.7 for the right arch leg on Span 4 as representative.

**Figure 4.6** Response N-Φ Hystereses for Left Arch Leg on Span 4

**Figure 4.7** Response N-Φ Hystereses for Right Arch Leg on Span 4
From Figure 4.4 and Figure 4.6 for the left arch leg on Span 4, it can be found that the difference by considering or by neglecting the M-N interaction are not significant. On the contrary, from Figure 4.5 and Figure 4.7 for the right arch leg on Span 4, it can be observed that the difference by considering or by neglecting the M-N interaction are relatively noticeable. This is probably resulted from the occurrence sequence of the yield of the member, especially whether the yield occurs under an axial load greater or smaller than the axial load when subjected to only dead load.

Therefore, the response on the left arch leg and the right leg on Span 4 will be discussed separately.

The detailed response moment-curvature-axial load hysteresis (M-Φ-N) at bottom of right arch leg of Span 4 (AL 4 R) is summarized for the most important time period (approximately from 37 s to 41 s). The curvature history and moment history is shown in Figure 4.8 (a) and (b) respectively, followed by M-Φ hysteresis in Figure 4.9, and the interaction of M-N and N-Φ hystereses are shown in Figure 4.10 and Figure 4.11, with the comparison between the cases in which the M-N interaction of arch leg is considered or neglected.

![Figure 4.8 Response Histories of Curvature and Moment for Right Arch Leg on Span 4](image)
Here, three important time points are detailed illustrated: [1] 37.77 s, when AL-4-R reaches yield in the case that considers the M-N interaction; [2] 40.74 s, when AL-4-R reaches yield in the case that neglects the M-N interaction; [3] 40.77 s, when AL-4-R reaches ultimate in the case that considers the M-N interaction. The history during the time span from 37.0 s to 41.0 s is shown as representative.

Figure 4.9 Response Moment-Curvature (M-Φ) Hystereses of Right Arch Leg on Span 4

Figure 4.10 Response Moment-Axial Load Hystereses of Right Arch Leg on Span 4
It can be found in Figure 4.8 (a) and (e) that by axial load due to only dead load ($N_{\text{dead}} = 1396$ kN), the ultimate curvature of arch leg ($\Phi_{u,\text{dead}}$) is 0.0086 1/m. At [1] 37.77 s, the response curvature by considering the M-N interaction reaches the yield curvature due to smaller flexural resistance under smaller axial load (297 kN, about 21% of $N_{\text{dead}}$) at that time point. Therefore, the response curvature from [1] to [2] is generally greater in case considering M-N interaction than neglecting M-N interaction (in Figure 4.8 (a), Figure 4.9 and Figure 4.11) because the earlier yield results in more obvious residual flexural deformation by considering M-N interaction, while the response moment from [1] to [2] is generally smaller in case considering M-N interaction than neglecting M-N interaction (in Figure 4.8 (b), Figure 4.9 and Figure 4.10). Then, AL-4-R reaches yield at [2] 40.74 s by neglecting M-N interaction. At this time point, the response curvature by considering the M-N interaction (0.0079 1/m) is already as 3.6 times great as the response curvature by neglecting the M-N interaction (0.0022 1/m). After that, flexural response increases in both cases and ultimate stage is reached at [3] 40.77 s under axial load of 1048 kN (about 75% of $N_{\text{dead}}$).

Also from Figure 4.11, it can be observed that the flexural response of AL-4-R by considering the M-N interaction exceeds the N-$\Phi_u$ curve significantly, probably due to the earlier yield under low axial load. On the other hand, the response curvature of AL-4-R by neglecting the M-N interaction does not exceed the ultimate curvature of arch leg ($\Phi_{u,\text{dead}}$) of 0.0086 1/m until the end of analysis.

Furthermore, the response of bottom of left arch leg of Span 4 (AL-4-L) is summarized in
Figure 4.12 for the curvature history and in Figure 4.13 for the calculated ultimate ratio based on following Eq. (4.1) as well. The history during the time span from 40.0 s to 43.0 s is shown as representative.

\[ \gamma = \frac{\Phi}{\Phi_u} \]  
(4.1)

where,
\( \Phi \) : response flexural curvature;
\( \Phi_u \) : the ultimate curvature (considering M-N interaction or neglecting M-N interaction separately).

From Figure 4.12, we can see that the response curvature history is almost same by considering or neglecting the M-N interaction, since the yield has been reached under similar axial load condition in both cases. However, due to the fluctuation of axial load in arch leg, the ultimate curvature fluctuates notably (shown as the gray dotted line).

In Figure 4.13, the ultimate ratio is defined as the result that the response curvature divided by
the ultimate curvature varied with the response axial load at any time point for the case in which the M-N interaction is considered, and the result that the response curvature divided by the ultimate curvature ($\Phi_u$-dead) under the axial load due to only dead load for the case in which the M-N interaction is neglected. At 40.78 s (in Figure 4.12), the response curvature by considering the M-N interaction exceeds the ultimate curvature (0.0077 1/m, about 90% of $\Phi_u$-dead) under axial load of 1639 kN (about 117% of $N_{\text{dead}}$). About 1.04 times of the ultimate curvature (defined as the ultimate ratio) is reached, as shown in Figure 4.13.

This suggests that AL-4-L reaches the ultimate stage by considering the M-N interaction because the ultimate curvature decreases under greater axial load than by only dead load. On the contrary, by neglecting the M-N interaction, maximum ultimate ratio of 0.994 is got, and the ultimate curvature of arch leg ($\Phi_u$-dead) of 0.0086 1/m has not been reached until the end of analysis.

4.3.3 Summary of analytical results

To sum all results up, the ultimate ratio at bottom of all arch legs (AL-3-L and AL-3-R for Span 3, AL-4-L and AL-4-R for Span 4) are calculated similarly to that shown in Figure 4.13. The maximum ultimate ratio at these 4 points are plotted in Figure 4.14, with the comparison between that by considering or neglecting the M-N inter-action for arch legs.

![Figure 4.14 Comparison of Ultimate Ratio of Arch Leg by Considering or Neglecting M-N Interaction](image)

It can be found that for the case in which the M-N interaction is considered, all 4 points have
the ultimate ratio greater than 1.0, suggesting that the ultimate stage is reached at all 4 points. The greatest value is 2.237 at AL-4-R, where the yield occurred very early due to decrease of yield resistance by decrease of response axial load. Besides, the average ultimate ratio of 4 points is 1.521 by considering M-N interaction (in Figure 4.14).

On the other hand, for the case in which the M-N interaction is neglected, only 1 of 4 points (AL-3-L) exceeds ultimate stage slightly, with the ultimate ratio being 1.063. All ultimate ratios of other 3 points (respectively 0.978, 0.994 and 0.767) are smaller than 1.0, do not reach ultimate stage. Furthermore, average ultimate ratio is 0.950 neglecting M-N interaction, which is smaller by 38% than that considering M-N interaction (1.521).

As a consequence, it can be concluded that no matter the yield occurs to section under axial load greater or smaller than the axial load by only dead load, the maximum flexural response would be underestimated if neglecting the M-N interaction, mainly because of the obvious decrease of ultimate curvature caused by increase of axial load.

4.4 Possible Design Measures

For the structural characteristics of Xiaoyudong Bridge, it should be noticed that the ties are few (diameter of 8 mm and spacing of 200 mm), and the sectional area of arch leg is relatively small (b*d = 720 mm * 350 mm), as illustrated in Figure 4.15.

![Figure 4.15 Original Section and Assumed Sections for Arch Leg](image-url)
Taking it into consideration that the flexural ductility and the flexural/axial capacity can be improved by adding ties, and by enlarging the sectional area, two types of section are assumed. The assumed section I applies twice of ties than the original section, by half of the original tie spacing (100 mm), while the assumed section II uses the 1.2 times of height and width of section (1.2b*1.2d = 864 mm * 420 mm, the sectional area becomes 1.44 times of the original sectional area).

Therefore, the calculated N-Φu curves of the original section (solid line in gray), the assumed section I (dotted line in pink) and the assumed section II (dash-dot line in green) of arch leg, and the response range at AL-4-R and AL-4-L are summarized in Figure 4.16.

![Figure 4.16 Possible Design Measure to Avoid Ultimate Stage of Arch Leg](image)

It can be found that the response range of both points exceeds the ultimate criterion (N-Φu curve) of the original section, but lays inside the ultimate criteria (N-Φu curve) of assumed section I and II. Furthermore, since the greater ductility and capacity would probably reduce the response, the response range would be even smaller and the section would be safer. This indicates that the severe failure at bottom of arch leg, the key member of this bridge, should have been possibly avoided by adding the ties volume or by enlarging the sectional area.

### 4.5 Summary

In this chapter, the influence due to M-N interaction was discussed by performing the nonlinear dynamic analysis with or without the consideration of the M-N interaction for the main
supporting member, arch legs, by applying the collapsed Span 3 and Span 4 as representative. As a consequence, following conclusions have been drawn:

(1) Based on evaluation on influence of M-N interaction, it was found that by neglecting M-N interaction, flexural response was underestimated. Maximum ultimate ratio was greater by considering M-N interaction, no matter the first yield occurred under an axial load greater or smaller, compared to the axial load by only dead load. Average maximum ultimate ratio of 4 points (1.521) was greater if considering M-N interaction, by about 38% than neglecting that (0.950). Especially when yield occurred early due to smaller axial load than that under only dead load, maximum ultimate ratio (2.237) in case with M-N interaction would be as 2.9 times great as that in case without M-N interaction (0.767). Thus, subjected to extremely high axial load (about 65% of axial capacity), arch legs on Span 3 & 4 suffered extensive failure, indicating that the arch legs, as the main supporting member, had too small sectional area and too few confining reinforcement.

(2) For future design of this bridge type, the RC rigid-frame arch bridge, it is possible to apply large sectional area, or to apply more confining reinforcement. This design measures can significantly improve the ductility of the arch leg. By doing so, the severe failure at bottom of arch leg, the key member of this bridge, should have been possibly avoided by adding the ties volume or by enlarging the sectional area.

References

Chapter 5 Evaluation on Seismic Behavior of RC Columns based on E-Defense Excitation Tests

5.1 Introduction

In 1995 Kobe Earthquake, a number of bridges collapsed because the RC columns which supported decks of bridges failed in flexure at base. It is known that the main cause of the flexural failure at base is lack of flexural capacity as well as ductility capacity resulted from underestimated seismic lateral force, insufficient amount of ties and inadequate development of the lap-splice of ties. Almost all columns failed in Kobe Earthquake were built in 1970s and designed based on the 1964 Design Specifications of Steel Road Bridges \[5.1\]. A number of bridges built in 1970s were retrofitted after Kobe Earthquake. However, the bridges, which have similar properties with the column failed in Kobe Earthquake and are not retrofitted, still exist in other area where strong earthquake may hit on. Therefore, it is important to investigate the failure mechanisms of the columns failure in Kobe Earthquake. Consequently, to study the mechanisms for a large scale reinforced concrete column representing typical columns built in the 1970s, namely C1-1 \[5.2\] (in Figure 5.1), the first shake table experiment using E-Defense was conducted in December 2007.

![Figure 5.1 Experimental Test on C1-1 Specimen](image-url)
On the other hand, it was reported \(^5\) that the RC columns also suffered significant residual displacement although the apparent damage might be not notable in 1995 Kobe Earthquake, causing the difficulty or impossibility to retrofit. Then, in 2002 Specification \(^4\), it was defined that the residual displacement should not exceed 1% of the height of pier to ensure the safety. However, only few studies were published focusing on the residual displacement of RC column of bridge. For study on actual damage of RC column in 1995 Kobe Earthquake \(^3\), general damage condition was summarized, and it was concluded based on numerical analyses that the E-W component of Takatori wave might induce only small hysteresis residual response. Besides, other studies (example as Ref. \([5.5]\)) based on experimental tests, only discussed the hysteresis residual response of single RC column. Thus, the influence from neighboring structure, such as that from the frictional resistance on side bents, has not been studied by comparing experimental result and analytical result.

Furthermore, numerical analyses based on fiber model were performed by Ukon, et al. \(^2\) and by Kawashima, et al. \(^6\) to reappear the experimental test of C1-1. In these studies, however, the attention was mainly paid on peak response and general damage condition. The analysis in this study, on the other hand, is performed to simulate the residual response and to evaluate its mechanisms.

In this study, aiming at studying the residual behavior of C1-1, Case 1 of dynamic analysis with frictional coefficient of 0.12 (based on element test on movable bearing) is conducted and compared with experiment. After the discussion of its correlation, the mechanisms of how the residual displacement occurred is discussed. Furthermore, to clarify if the free assumption of friction (based on specification \([5.4]\)) is reasonable, Case 2 (\(\mu=0.00\)) is performed and evaluated in detail. This evaluation concentrating on C1-1, the RC column specimen that failed in flexure, is presented in Section 5.2 ~ Section 5.5.

On the other hand, to reduce amount of longitudinal reinforcement at the section where it is not necessary by distributed moment, it was common practice until the mid-1980s to cut off LG bars in RC columns. However, this type of piers was extensively damaged during 1978 Miyagi-ken-oki Earthquake, 1982 Urakawa-oki Earthquake and 1995 Kobe Earthquake. Its failure mechanisms were widely studied. Study \([5.7]\) proposed an inspection method to identify its vulnerability, focusing on effect by development length. It was understood that this type of column failed without sufficient development length. The shear resistance of this type of columns is normally determined by concrete and confinement (hoops for pier) separately \([5.8]\). The resistance by concrete is got by assuming the shear develops along 45° to the vertical axis. For RC columns with cut-off LG bars, however, lateral cracks due to flexure may occur firstly, and leads to further shear cracks. This phenomenon may further cause the degrading of shear resistance by concrete and earlier failure of the columns with cut-off of LG bars.

Besides, study \([5.9]\) based on experimental tests showed that the scale effect had notable
influence on its behavior and failure mechanisms. In addition, detailed progress of failure, relation between flexural failure and shear failure, mechanisms of shear resistance, especially at cut-off position, were still not able to be explained clearly. The full scale experimental test on RC column with cut-off LG bars (specimen No. C1-2, as shown in Figure 5.2) by E-Defense excitation provides researchers the opportunities to understand the failure mechanisms close to the actual failure.

![Figure 5.2 Experimental Test on C1-2 Specimen](image)

Therefore, in this study, the failure mechanisms, especially the progress of damage focus on the cut-off position and the degrading of shear resistance of C1-2 will be evaluated in details. This evaluation concentrating on C1-2, the RC column specimen that failed in shear, is presented in Section 5.6 ~ Section 5.8.

### 5.2 Experimental Setup and Analytical Condition for C1-1

As mentioned in Section 5.1, C1-1 is the specimen that was designed to damage by flexural moment as a typical column which was built in the 1970s. In this section, the setup of C1-1, the element test result of its movable bearing, and the analytical modeling will be explained in details.

#### 5.2.1 C1-1 specimen and experiment setup

A large scale bridge experimental program was started in 2005 as one US-Japan cooperative
research programs based on NEES (the George E. Brown, Jr. Network for Earthquake Engineering Simulation) and E-Defense collaboration. Component models (C1 series) and system models (C2 series) were planned \cite{5.2}. The purpose of C1 series experiments is to clarify the failure mechanism of single RC columns using full scale models. In C1 series, C1-1 column, as shown in Figure 5.1, is a typical column which was built in the 1970s. Its collapse is mainly caused by flexural failure, as a representative damage type in 1995 Kobe Earthquake. Following the preliminary experiments and analyses, excitation experiment of C1-1 (Figure 5.1) was conducted in December 2007.

C1-1 is a 7.5 m tall, 1.8 m diameter reinforced concrete column. The general drawing of C1-1 is shown in Figure 5.3.

![Figure 5.3 C1-1 Specimen](image)

C1-1 was anchored to E-Defense shake table by a 1.8m thick square footing. It was designed as a full scale model specimen based on a combination of the static lateral force method and the working stress design (seismic coefficient method) which were specified in the 1964 Design Specifications of Steel Road Bridges \cite{5.1}. The lateral seismic coefficient of 0.23 and the vertical seismic coefficient of ±0.11 were adopted in the design. The seismic performance of C1-1 specimen in the longitudinal direction was evaluated based on the 2002 JRA Design Specification of Highway Bridges \cite{5.4}. Yield and ultimate displacement ($\delta_y$ and $\delta_u$) are 0.046 m and 0.099 m.

As shown in Figure 5.3, the column has 3 layers of longitudinal reinforcing bars with 29 mm diameter, respectively 32, 32 and 16 at outer, middle and inner layers. Deformed circular
stirrups with 13 mm diameter are provided at 300 mm interval, except outer ties at top 1.15 m zone and at base 0.95 m zone with 150 mm interval. Stirrups are lap spliced with 390 mm (30 times of its diameter). Consequently, longitudinal reinforcement ratio is 2.02%, and tie volumetric reinforcement ratio is 0.32% for middle and 0.42% for top or base. On the day of experiment, actual strength of longitudinal bars, stirrups and concrete were measured as 366 MPa, 193 MPa and 33 MPa.

The residual damage condition at base of column of C1-1 is shown in Figure 5.4, from which we can observe extensive damage of covering concrete and several buckling of longitudinal reinforcing bars.

Upon RC column and two steel bents, two simply supported steel decks were set. Four mass blocks were fixed on decks by anchorages to simulate the dead load. Between the deck and the column, 2 fixed bearing (fixed in both the longitudinal and the transverse directions) and 4 slider (free in both the longitudinal and the transverse directions) were set. Between the deck and bents at two ends, respectively 1 movable bearing and 2 sliders (same as that on column) were used.

The setup of the movable bearing on bents is shown in Figure 5.5. It was designed to provide the frictional force in the longitudinal direction (X axis), to be fixed in the transverse direction (Z axis), and free in the rotational direction.
Footing of C1-1 was fixed on shake table of E-Defense. The table was excited using E-Takatori ground motion (which was modified from observed Takatori ground motion, considering the soil-structure interaction by FEM analysis). Consequently, the resultant ground motion (E-Takatori ground motion) has about 80% amplitude of the original motion, without changing other periodical characteristics [5.2]. Main excitation using 100% E-Takatori ground motion was conducted twice.

5.2.2 Element tests on movable bearing

In C1-1 experiment, the response movable bearing on side bents was not measured. However, during summarizing the experimental results, the influence of frictional load of movable bearings (shown in Figure 5.5) was found significant.

In the experimental report [5.2], the frictional coefficient of these movable bearing was discussed numerically. Analytical results suggested the frictional coefficient being the value approximately between 0.1 and 0.2. However, only the peak response was correlated to obtain the possible frictional coefficient in the analytical study in the report, without taking other influences into consideration.

Aiming to accurately measure the frictional coefficient and to detailed understand the characteristics, element tests of the movable bearing were conducted [5.10] after the experimental test of C1-1, with identical specimens of movable bearing. The setup of testing equipment is shown in Figure 5.6.
Specimen of movable bearing was fixed between bed (connected to actuator) and axial load block. Total 2 specimens were tested respectively under 3 sets of parameters (contact pressure, excitation velocity, vibration amplitude). Example of the experimental result is shown in Figure 5.7.

This example shows the test on Specimen No. 1 with frequency of 1.50 Hz, contact pressure of 6.3 MPa (namely contact load of 450 kN) and amplitude of ±100 mm. During the test, the intersection of the P-δ hysteresis and Y-axis is taken as the nominal lateral load when sliding. Then, by averaging this lateral load of sliding from all cycles, the sliding load (P_y) was calculated as averagely 54.2 kN, and the corresponding displacement (δ_y) was 0.62 mm.
Therefore, the initial stiffness \((K)\) of this specimen can be calculated as 87.4 kN/mm \((= 54.2 \text{ kN} / 0.62 \text{ mm})\). On the other hand, the frictional coefficient \((\mu)\) can be calculated as 0.120 \((= 54.2 \text{ kN} / 450 \text{ kN})\).

As a consequence, initial stiffness and frictional coefficient for all tested cases are summarized in Figure 5.8 and Figure 5.9 respectively for all cases under parameters of contact pressure, excitation velocity, vibration amplitude.

![Figure 5.8 Measured Initial Stiffness in Element Tests](image1)

![Figure 5.9 Measured Frictional Coefficient in Element Tests](image2)
It can be observed in Figure 5.8, although the unevenness is slightly great since sliding load \( (P_y) \) and displacement \( (\delta_y) \) is very sensitive, the average initial stiffness is about 104 kN/mm. For the frictional coefficient shown in Figure 5.9, minimal value is got as about 0.06 when the contact pressure is extremely high (20 MPa), which is approximately 3 times as that shown in Figure 5.7 (6.3 MPa). Oppositely, the maximal value is obtained as nearly 0.16 when the vibration amplitude \((\pm 24 \text{ mm})\) is noticeably small as approximately 1/4 of that shown in Figure 5.7 \((\pm 100 \text{ mm})\). Since these extreme conditions were not reached in experimental test, the average value of the frictional coefficient (0.119) is considered to be suitable to be applied in numerical analysis. As a consequence, these measured characteristics of the movable bearing in the element tests (the initial stiffness of about 104 kN/mm and the frictional coefficient of about 0.120) will be applied in this study, rather than the idealized assumptions which have been utilized in the report \([5.2]\).

5.2.3 Analytical modeling and conditions

3D frame model is established for C1-1 specimen, for all column, decks and end supports as shown in Figure 5.10, by using RESP-T \([5.11]\).
Bearing is idealized as illustrated in Figure 5.11. Bilinear frictional spring (Figure 5.11 (a)) is utilized for movable bearing. According to the experimental results of element tests stated above in Sub-section 5.2.2, average initial stiffness \( K = 104 \text{ kN/mm} \) and average frictional coefficient \( \mu = 0.12 \) are utilized for model of Case 1. Besides, impact springs (Figure 5.11 (b)) are used for contact and separation at side sliders.

![Figure 5.11 Modeling of Bearings](image)

The moment-curvature (M-\( \Phi \)) relationship is assumed to the elements of column, while all other members are assumed as elastic elements. The stress-strain (\( \sigma \)-\( \varepsilon \)) relationship of concrete, according to Specification for Highway Bridges: Part V Seismic Design\(^{[5.4]} \), is shown in Figure 5.12.

![Figure 5.12 \( \sigma \)-\( \varepsilon \) Relation of Concrete based on Specification](image)

\[ \begin{align*}
\sigma & = \sigma_{cc} - E_{des}(\varepsilon_c - \varepsilon_{cc}) \\
\sigma_{cc} & = 33.0 \text{ MPa} \\
\sigma_c & = E_c \varepsilon_c \left(1 - \frac{1}{n}(\varepsilon_c - \varepsilon_{cc}) \right)^n \\
\varepsilon_{cc} & = 0.003 \\
\varepsilon_{ccl} & = 0.008 \\
\end{align*} \]
The actual strength ($\sigma_{cc}$) measured on experiment day of 33.0 MPa is used. Here, $\sigma_c$ and $\varepsilon_c$ means strain and stress of concrete; $\varepsilon_{cc}$ means strain of concrete when reaches the actual strength ($\sigma_{cc}$); $\varepsilon_{ccl}$ means strain of concrete when reaches the 0.8 times of actual strength ($\sigma_{cc}$); $E_c$ ($= 25.1$ GPa) stands for Young’s modules of concrete.

Furthermore, JR-RC relation (illustrated in Figure 5.13) is used for tetra-linear un-/re-loading hysteresis for column element (element at column base is shown as example).

![Figure 5.13 Unloading and Reloading Hysteresis Relation at Column Base according to JR-RC Model [5.12]](image)

As shown in Table 5.1, JR-RC hysteresis model [5.12 ~ 5.13] was proposed for tetra-linear hysteresis model according to the experimental tests on specimens with shear span ratio of 2.5 ~ 9.9, longitudinal bar ratio of 0.7% ~ 3.3%, stirrup ratio 0.1% ~ 2.3%, and axial ratio of 0.0 ~ 0.32. On the other hand, C1-1 has the characters of shear span ratio of 4.4, longitudinal bar ratio of 2.02%, stirrup ratio 0.42%, and axial ratio of 0.025. This suggests that JR-RC hysteresis model is suitable for the analysis of C1-1 Specimen, for not only the peak response but also the post-peak response and the residual response.

<table>
<thead>
<tr>
<th>Item</th>
<th>Shear span ratio ($l_a/d$)</th>
<th>Longitudinal bar ratio (%)</th>
<th>Stirrup ratio (%)</th>
<th>Axial ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ref</td>
<td>[5.9 ~ 5.10]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2.5~9.9</td>
<td>0.7~3.3</td>
<td>0.1~2.3</td>
<td>0.0~0.32</td>
</tr>
<tr>
<td>C1-1</td>
<td>4.4</td>
<td>2.02</td>
<td>0.42</td>
<td>0.025</td>
</tr>
</tbody>
</table>

Table 5.1 Applicability of JR-RC Hysteresis Model
For calculating of M-Φ relationship (hysteresis relationship shown in Figure 5.13), following definitions are applied in this study: (1) the crack stage (Point C in Figure 5.13) is defined as when surface concrete reaches its flexural strength; (2) the yield stage (Point Y in Figure 5.13) is defined as the longitudinal reinforcement reach the yield strength; (3) the ultimate stage (Point U in Figure 5.13) means the core concrete reaches its ultimate strain (εucc). Since the response in actual experiments exceeded the ultimate stage greatly, curvature Φ is assumed to increase continuously with steady moment after ultimate stage. Therefore, based on calculation, the moment and the curvature are got as 9.667 MNm and 1.692*10⁻³ 1/m for yield and 13.617 MNm and 1.002*10⁻² 1/m for ultimate stage. Although same strain-stress relations for material in former studies [5.2 - 5.6] are applied here, the M-Φ relationship and JR-RC model is directly used for evaluating the residual response of C1-1.

On the other hand, the unloading stiffness is evaluated by applying the stiffness degrading coefficient of -0.4 according to former studies [5.12 - 5.13]. Besides, since the displacement in experimental test by base rotation due to pull-out of longitudinal bars contributes to the total displacement by about 30% at 1δy (in normal range of RC columns), but decreases obviously with the increase of displacement, especially after δu, this effect is not considered in current analysis.

Furthermore, modal damping ratio of 0.1 % was assumed at 1 Hz and 25 Hz for Rayleigh damping, since the radiational energy dissipation of a column anchored to a shake table is extremely smaller than the real energy dissipation of a column embedded in the actual ground [5.14].

Response acceleration at column base by averaging the measured accelerations on shake table at four corners is input to the base of footing, based on the assumption that the footing is rigid, and to two bents as two sides. As being explained formerly in Sub-section 5.2.1, 100% E-Takatori ground motion was applied to C1-1 specimen twice. Only the 1st excitation is used in current analysis.

Wave forms on longitudinal and transversal directions are shown in Figure 5.14. It can be found that peak acceleration of 8.22 m/s² and 5.76 m/s² occurred at 6.62 s and 4.00 s for each direction. For calculation, Newmark-β (β = 1/4) method is applied in numerical integration, with the time interval being 1/1000 s.
5.3 Analytical Result of Case 1

In this section, the analytical results of Case 1 will be explained by compared with the experimental results. The peak response displacement in Sub-section 5.3.1, the flexural response in Sub-section 5.3.2, and the residual response in Sub-section 5.3.3 will be explained respectively.

5.3.1 Peak response displacement

Response displacement histories on top of column (7.5 m from base of column) are shown for the longitudinal direction in Figure 5.15 and the transverse direction in Figure 5.16, comparing the results by experiment and analysis.

Figure 5.14 Input Wave Forms

Figure 5.15 Comparison of Displacement Histories on Top of Column (longitudinal)
It can be observed in the longitudinal direction (shown in Figure 5.15) that response displacement on top of column (7.5 m from base of column) by the analytical result coincides with the experimental result very well, and the negative residual displacement in experiment is reappeared as well, although the positive peak is slightly overestimated. In the analysis, the positive peak and the negative peak is reached as +0.2063 m at 6.930 s (+0.1692 m at 6.910 s in experimental test) and as -0.1897 m at 7.620 s (-0.1710 m at 7.615 s in experiment) respectively.

On the other hand, for the transverse direction illustrated in Figure 5.16, the top of column deforms maximum +0.1453 m at 7.315 s (+0.1335 m at 7.380 s) to the positive side and -0.0877 m at 8.890 s to the negative side (-0.1187 m at 9.015 s). Obvious difference occurs from negative peak. It is found the post-ultimate resistance drops notably in TR direction in experiment. Since this drop does not occur to LG direction, the post-ultimate resistance is assumed to be identical in current study. This is the possible reason of the error after negative peak in transverse direction. Thus, response displacement by analytical result shows comparatively good agreement with experimental result for the positive peak, but less good after reaching the negative peak.

Table 5.2 Comparison of Peak Displacement

<table>
<thead>
<tr>
<th>Displacement (m)</th>
<th>Experiment</th>
<th>Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp</td>
<td>Error</td>
</tr>
<tr>
<td>LG</td>
<td>+</td>
<td>+0.1692</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-0.1710</td>
</tr>
<tr>
<td>TR</td>
<td>+</td>
<td>+0.1335</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-0.1187</td>
</tr>
</tbody>
</table>
To sum it up, comparison of peak response displacement on top of column is illustrated in Table 5.2. In longitudinal direction, +21.9% and +10.9% errors suggest acceptability of the analysis, although the response displacement is slight overestimated. On the other hand, in transverse direction, errors occur especially for the negative peak (-26.1%) while the positive peak is reappeared well with only +8.8% error. These indicate that vibration is generally overestimated, but the general trend is approximately simulated. Although response in transverse direction was reappeared not well after the negative peak, the response in longitudinal direction, especially the residual response was well simulated. Thus, the influence by friction on residual response (in following) can be discussed based on analytical result in longitudinal direction.

5.3.2 Flexural response

After assessing the response displacement on top of column, the flexural behavior of column is going to be introduced in this section. M-δ hystereses, curvature distribution and curvature histories at representative sections will be explained as following by comparing the analytical result and experimental result.

Firstly, M-δ hystereses are plotted in Figure 5.17 for the longitudinal direction and in Figure 5.18 for the transverse direction. Here, moment at bottom section of column and displacement on top of column are used for comparison.

![Figure 5.17 Comparison of Response M-δ Hystereses in Longitudinal Direction](image)

We can see that for the LG direction (Figure 5.17), hystereses coincide with each other, with slightly overestimated displacement in analysis. For example, the positive peak point a in experiment and a’ in analysis, with the moment is got almost same (14.7 MN•m and 14.8 MN•m)
with 21.9% different displacement mentioned before. For other two pairs of key point b and b’ (when moment returns to zero), and c and c’ (the negative peak), only the displacement is overestimated as well. The unloading stiffness from a to b and from a’ to b’ are very similar, although earlier deflection of the curve in experiment (a to b) causes particular error between b and b’. Thus, the M-δ hysteresis by analysis roughly encloses that by experiment.

**Figure 5.18 Comparison of Response M-δ Hystereses in Transverse Direction**

Furthermore, for the transverse direction (shown in Figure 5.18), M-δ hysteresis in experimental test is simulated by our analysis with acceptable difference. In the experiment, shown as the black solid line, notable resistance decrease due to reloading occurred to both the positive and the negative side in transverse direction (no similar phenomenon of this resistance decrease was observed in longitudinal direction). However, since this phenomenon was not taken into consideration, both response moment and response displacement keep developing from previous cycles when reloading. Therefore, for the positive peak point d in experiment and d’ in analysis, the moment is as 14.1 MN·m in analysis but only 12.1 MN·m in experiment. Although the next point e and e’ (when moment returns to zero) have limited difference, error for negative peak point f and f’ is still obvious (13.1 MN·m in analysis but only 10.7 MN·m in experiment). Though, general phenomenon is reappeared roughly.

Besides, the analytical correlation is evaluated based on response curvature as well. The comparison of the curvature distribution at when the top of column suffers maximum displacement in longitudinal direction (6.910 s in the experiment and 6.930 s in the analysis) is shown in Figure 5.19.
Figure 5.19 Comparison of Curvature Distribution along Height of Column from Base (longitudinal direction)

In Figure 5.19, the curvature distribution from base to 1.8 m of column is plotted because the curvature was only measured at this range. Here, the curvature in experiment is obtained from relative displacement measured in experiment. It can be found that analytical result provides a comparatively smoother curve of curvature distribution, with its general trend similar to that by experimental test.

However, at column base, the experiment shows curvature significantly greater than that by analysis. The curvature of 0.0678 1/m at 0.04 m high from base in experiment is about twice as the curvature of 0.0349 1/m at 0.00 m high from base in analysis. Attention should be paid that the pull-out effect of longitudinal reinforcing bars is not taken into consideration in current analysis. In the experimental test, however, the relative displacement contributed due to the pulling out of longitudinal bars from the footing might be notable [5,6].

To confirm this in details, curvature in longitudinal direction at to upper surface (at heights of 0.18 m and 0.38 m from base) are shown in Figure 5.20 and Figure 5.21 as representative.

We can see that curvature history at 0.38 m (shown in Figure 5.20) in analysis well coincides with experimental result. Besides, at 0.18 m from base (shown in Figure 5.21), curvature is very similar to each other. For the peak response at this section, the curvature reaches at -0.0288 1/m and +0.310 1/m in analysis and -0.0244 1/m and +0.0181 1/m in experiment, suggesting good coincidence.
5.3.3 Residual response

The peak response and the flexural response have been explained above. At last, residual response in longitudinal direction, in terms of displacement and curvature distribution, will be discussed in this section.

Formerly, Figure 5.15 showed response displacement histories in longitudinal direction. Point g and g’ respectively the residual displacement in experiment and Case 1. It can be found residual displacement was -0.0206 m in experiment and -0.0139 m in Case 1. The residual displacement in analysis is smaller than that in experiment by 32.5%. Besides, these residual displacements are respectively 6.7% and 12.0% of each maximum displacement. The ratio in experiment is greater, even exceeding 10%.

Furthermore, Figure 5.22 shows comparison of the curvature distribution at 30.0 s (namely residual curvature distribution) in longitudinal direction. It can be observed that from base to 0.5 m high, the distribution in Case 1 roughly coincides with experiment, although it is obviously smaller in analysis (curvature at base is -0.0025 1/m in Case 1 and -0.0078 1/m in experiment).
For height from 0.5 m to 1.0 m, the max curvature in experiment for this height is noticeable (0.0083 1/m) at about 0.8 m high. However, Figure 5.23 shows the crack condition. It can be observed that two cracks crossed the fixed equipment at Point h and i for curvature measurement. Thus, great curvature occurred at these points. Therefore, although residual displacement was got with slightly smaller value in analysis, residual curvature distribution was roughly reappeared at base.

Figure 5.22 Residual Curvature Distribution

Figure 5.23 Crack Condition on East Side
To sum it up, based on the dynamic analysis of Case 1 ($\mu = 0.12$), the response displacement on top of column (height of 7.5 m from base), the moment acting on base of column, and residual curvature distribution were approximately reappeared for the experimental test, although there was still particular difference for the curvature distribution and the acceleration distribution along column. Thus, flexural behavior and residual response of this column was roughly simulated based on Case 1.

5.4 Mechanisms of Residual Displacement

Analytical result of Case 1 ($\mu = 0.12$) has been explained generally in Section 5.3. In this section, the mechanisms of the residual displacement will be evaluated. In study [5,3] of residual displacement of bridge column in 1995 Kobe Earthquake, notable residual displacement was abundantly observed, but based on the numerical analyses, it was inferred that the E-W component of Takatori wave may result in limited residual displacement of RC column, compared with Amagasaki wave. This is possibly caused by the discussion based on only single column ignoring neighboring structure. In current study of C1-1 (with two side bents), the residual displacement is notable (-2.06 cm in experiment and -1.39 cm in Case 1 of analysis).

In addition to hysteresis residual response of single column, other factors (such as residual force due to frictional resistance passed by deck from side bents) may also have influence. This contrast, between the notable residual displacement in actual experiment of C1-1 and the characteristics of Takatori wave, makes the discussion on the possible influence from neighboring structures necessary.

Here, a 10.0 s-long wave form of 0-acceleration is added at the end of the original 30.0 s-long wave form (shown in Figure 5.14) in the analysis, to let the vibration of structure settle down. The response acceleration histories on top of column along the LG direction at end of analysis are shown in Figure 5.24.

![Figure 5.24 Comparison of Response Acceleration Histories on Column Top (Longitudinal Direction)](image-url)
It can be confirmed that the response acceleration on column top is reduced to no greater than ±10 gal in the last 2.0 s by this method. Due to fast settling down, the residual in experiment is defined as the measured values at 30.0 s. On the other hand, to evaluate not only the residual displacement, the residual curvature, but also the residual load (which will be applied to explain the mechanisms based on analytical result in the following sections), it is necessary to obtain very steady values. Therefore, the residual in analysis is defined as the average of the last 2.0 s. In following, the discussion is going to be made based on two main reasons, by hysteresis and by friction, separately in Sub-section 5.4.1 and Sub-section 5.4.2.

5.4.1 Residual displacement due to hysteresis

In order to discuss the residual response due to hysteresis, the response M-Φ hystereses at column base along the LG direction is compared in Figure 5.25 between experiment and analysis. Here, the curvature at 4.0 cm from column base in experiment is got based on the relative displacement between heights 0.0 cm and 8.0 cm (lowest position measured by LVDT), while the curvature at nearest height (0.0 cm) in analysis is used for comparison (rather than that at 4.0 cm considering the ignorable difference).

![Figure 5.25 Comparison of Response M-Φ Hystereses (Longitudinal Direction)](image)

It can be observed from Figure 5.25 that the M-Φ hysteresis in experiment is roughly reappeared by analysis since the values at peak and X-axis are similar to each other. Thus, the general flexural response in experiment is roughly reappeared by analysis. Furthermore, the points at the peak response in both positive (points j and j’ for experiment and analysis respectively) and negative (points l and l’), and at when moment decreases until 0 (namely the temporary residual curvature, points k and k’, and m and m’) are compared in details.
Firstly, for the positive direction, the peak response curvature is 0.035 1/m in analysis at point j’, which is about 16.7% smaller than that in experiment (0.042 1/m at point j). Then, when the flexural moment decreases to 0 during the unloading procedure, the M-Φ curve crosses the X-axis with the curvature of 0.027 1/m in analysis at point k’, which is about 42.1% greater than that in experiment (0.019 1/m at point k).

On the other hand, for the negative direction, the minimum is reached at -0.037 1/m in analysis at point l’. This is about 7.5% smaller than the minimal curvature (-0.040 1/m at point l) in experiment. During the following reloading procedure, the M-Φ curve crosses the X-axis with the curvature being -0.029 1/m in analysis at point m’, greater (absolute value) than the curvature -0.020 1/m at point m in experiment by about 45.0%.

To sum it up, the positive and negative peak curvature is generally smaller in analysis by about 12.1% (average of 16.7% for positive peak and 7.5% for negative peak) than in experiment. However, when the moment decreases to 0, the temporary residual curvature (points k’ and m’ compared to points k and m) is averagely greater in analysis by about 43.6%. This indicates that the temporary residual response is more likely to occur in analysis compared with that in experiment, since the response curvature decreases faster during unloading in analysis.

From another point of view, the difference between the negative and the positive peak curvature is -5.4% in analysis (points j’ and l’) and +5.0% in experiment (points j and l), and the difference between two temporary residual curvature is -6.9% in analysis (points k’ and m’) and -5.0% in experiment (points k and m) respectively. This suggests that the response of this RC column under the E-W component of Takatori wave is noticeably balanced between the positive and the negative, since the differences are all smaller than 10%.

Therefore, the conclusion can be drawn that the residual response due to hysteresis is not significant.

5.4.2 Residual displacement due to friction

In this sub-section, the possible influence on residual response due to the frictional force on side bents will be discussed.

The force condition of the system by deck, column and side bent (west as an example) is illustrated in Figure 5.26. When deck does not move, the forces by movable bearing (F_{f}) on side bents and by fixed bearing (F_{LG}) on the column should provide equal resistance. If the frictional coefficient is set as 0, or the movable bearing is replaced by free bearing (such as roller bearing), force by fixed bearing (F_{LG}) will be 0, since no force can be provided by movable bearing (F_{f}). However, for current Case 1 (μ=0.12) C1-1 specimen, residual force by fixed bearing (F_{LG}) may occur due to possible residual frictional resistance by movable bearing.
Therefore, to confirm the forces by movable bearing and fixed bearing, the response history of lateral force are plotted on movable bearing and fixed bearing in Figure 5.27 (those on west deck are shown as examples).

It can be observed that without input acceleration after 30.0 s, both forces on movable bearing and fixed bearing become steady gradually, although small vibration with high frequency still exists. By defining the residual force as the average of the last 2.0 s, the residual forces can be got as -9.62 kN for fixed bearing and 12.25 kN for movable bearing on the west deck. Similarly, those residual forces (4.60 kN, -3.87 kN) on the east deck can be calculated as well, and comparison is summarized in Figure 5.28. As a consequence, the residual force acting on column top (on fixed bearing) is about 80% (absolute values) of the residual frictional force (on movable bearing).
For discussing how these residual lateral forces (by fixed bearing on column top and movable bearing on side bents) occurs, the response P-δ hysteresis in analysis is plotted in Figure 5.29, with the one on west bent being the example.

Here, positive direction stands for the condition in which the movable bearing provides resistance toward east (right in Figure 5.30). Correspondingly, the mechanisms are summarized in Figure 5.30 (the deformation of side bent is not shown here since it responses only elastically).
At the start of sliding ((a) in Figure 5.29 and Figure 5.30), when deck moves towards west (left side) and movable bearing begins to slide, the resistance provided by frictional force is $F_{\text{fri}} = F_Y = 47.45$ kN (refer to model of bearing in Figure 5.11). Only elastic shear deformation of about 0.05 cm occurs to the movable bearing. From this time point, the relative displacement (namely the sliding displacement between movable bearing and deck) starts to occur.

Then, for maximum displacement towards west ((b) in Figure 5.29 and Figure 5.30), the frictional resistance by movable bearing stays same with that of (a). However, the relative displacement of about 21.28 cm develops noticeably between deck and movable bearing (the total displacement 21.33 cm minus the elastic shear displacement of 0.05 cm in condition (a)). On the other hand, the fixed bearing on the RC column resists most of the lateral load from inertial force of deck.

Following, with following vibration, the relative displacement ((c) in Figure 5.29 and Figure 5.30) at movable bearing cannot decrease to 0 and remains, leading to the remaining of frictional resistance. To balance this force, a counterforce from fixed bearing on column will act on deck. Thus, when deck settles down at last, it will be located to slightly left to the original position, because the residual force by movable bearing ($F_{\text{fri-res}}$) and by fixed bearing in LG direction ($F_{\text{LG-res}}$) remain. The counterforces of these two residual forces lead to the balance of
deck, and thus should have same absolute value but opposite direction. Consequently, this residual horizontal load ($F_{LG-res}$) acting on fixed bearing on column top also contributes to the residual displacement on column top.

### 5.4.3 Comparison of different behaviors of residual response

The residual responses due to flexural hysteresis and due to residual frictional force from movable bearing on side bents have been separately illustrated in Sub-section 5.4.1 and Sub-section 5.4.2. Thus, the comparison between them and their contribution ratio, based on the analytical result, will be summarized in this sub-section.

Since the residual displacement on column top is dominated by the residual curvature, the residual flexural response is discussed in details. Firstly, the response $M$-$\Phi$ hysteresis at base in the last 0.2 s (39.8 ~ 40.0 s) is plotted in Figure 5.31.

- **Point A**: $M_{UD-res} = -27.5$, $\Phi_{UD-res} = -0.338 \times 10^{-3}$
- **Point B**: $M_{LG-res} = -101.3$, $\Phi_{LG-res} = -1.507 \times 10^{-3}$
- **Point C**: $M_{UD-res} = -27.5$, $\Phi_{UD-res} = -0.338 \times 10^{-3}$

---

**Figure 5.31 M-$\Phi$ Hysteresis at Last 0.2 s (39.8-40.0 s) and Corresponding Unloading and Reloading Curve**

The residual point (at 40.0 s) is marked as Point A, and the residual moment and curvature is -4.5 kNm and -2.243$\times$10$^{-3}$ 1/m respectively. This residual curvature is approximately 1.33 times of the yield curvature ($\Phi_y = 1.692 \times 10^{-3}$ 1/m). It should be noticed that, both the flexural hysteresis and the residual loads acting on the column top may result in this residual response. Thus, the corresponding unloading and reloading curves based on the hysteresis relation of JR-RC model (refer to Figure 5.13) is shown in Figure 5.31 as well. It reaches the X-axis with
same stiffness before the residual Point A, and then turns to point at the experienced positive peak point.

To determine the actual contribution of residual loads, their residual condition is illustrated in Figure 5.32 (a).

At the end of analysis, residual horizontal load \( F_{\text{LG-res}} = -13.5 \text{ kN} = (-3.87) + (-9.62) \text{ kN} \), sum of west and east refer to Figure 5.28) acts on column top towards left, and residual vertical load \( F_{\text{UD-res}} = -1976.2 \text{ kN} \) acts on column top downwards. By respectively multiplying the height of column and the horizontal displacement by the loads, the flexural moment at column base due to these two loads can be calculated as \( M_{\text{LG-res}} = -101.3 \text{ kNm} \) (stands for the moment caused by residual lateral load on column top, which is induced by frictional force from movable bearing shown in Figure 5.26 ~ Figure 5.30) and \( M_{\text{UD-res}} = -27.5 \text{ kNm} \) (represents for the moment due to P-\( \delta \) effect).

Consequently, as illustrated in Figure 5.31, the corresponding point considering the residual loads can be got, by subtracting the flexural moment (shown in Figure 5.32 (a)) from the residual moment at Point A along the unloading and reloading curve. For example, the influence of moment by residual vertical load \( M_{\text{UD-res}} = -27.5 \text{ kNm} \) is got by subtracting this moment from total residual moment \((-4.5 - (-27.5) = 23.0)\). At the same time, curvature can be got by the unloading and reloading relations. By this method, Point B (considering influence by residual vertical load) and Point C (considering influence by residual horizontal load) can be
plotted as shown in Figure 5.31.

Thus, the residual curvature distribution and the residual curvature (at base) by flexural hysteresis, by residual horizontal load ($F_{LG\text{-res}}$) and by residual vertical load ($F_{UD\text{-res}}$) are plotted in Figure 5.32 (b). It can be found the residual curvature by flexural hysteresis, by residual horizontal load ($F_{LG\text{-res}}$) and by residual vertical load ($F_{UD\text{-res}}$), is respectively $\Phi_{\text{hysteresis}} = -0.398\times10^{-3}$ l/m, $\Phi_{LG\text{-res}} = -1.507\times10^{-3}$ l/m, and $\Phi_{UD\text{-res}} = -0.338\times10^{-3}$ l/m. Thus, they respectively contributes to the total residual curvature at base ($\Phi_{\text{total}} = -2.243\times10^{-3}$ l/m) by 17.7%, 67.2% and 15.1%.

On the other hand, since the curvature distributes along the height of column almost as triangle, and the curvature at base is significant, this great curvature at base dominates the displacement on top. As a result, the flexural hysteresis, the residual horizontal load ($F_{LG\text{-res}}$) and the residual vertical load ($F_{UD\text{-res}}$) may contribute to the residual displacement on column top with similar contribution ratio to the residual curvature at base stated in Figure 5.32 (b). Thus, the residual displacement caused by the E-W component of Takatori wave and the vertical load are slight. On the contrary, the residual displacement caused by the horizontal load acting on column top, which is resulted from the frictional force of movable bearing on side bents, is relatively obvious.

Therefore, residual horizontal load on column top by considering the friction resistance from the neighboring structure (the side bents for C1-1), has noticeable influence on the notable residual displacement.

### 5.5 Evaluation on Friction-Free Assumption for Movable Bearing based on Specification

As the result of Case 1 ($\mu=0.12$) explained above, the experimental result was roughly reappeared, and the mechanisms of residual displacement has been evaluated in details. It was found that the residual horizontal load on column top due to friction was essential to the residual displacement. On the other hand, in actual design procedure, the frictional force of movable bearing is usually ignored according to specification\[^5\]. By doing so, the horizontal load on column top and the response moment at column base would be overestimated, leading to relatively safer assessment of peak response displacement. However, the influence on residual displacement due to this friction-free assumption has not been discussed.

Since the residual displacement is equally critical to the workability of a RC column after the earthquake, Case 2, in which the frictional coefficient $\mu$ is set as 0.00 by applying the perfect free assumption according to specification\[^4\] as shown in Table 5.3. After introducing the general results in Sub-section 5.5.1, discussion on residual displacement will be presented in Sub-section 5.5.2.
### Table 5.3 Analytical Cases and Assumption

<table>
<thead>
<tr>
<th>Cases</th>
<th>$\mu$</th>
<th>Consideration and Assumption</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>0.12</td>
<td>Average value according to element test of movable bearing</td>
</tr>
<tr>
<td>Case 2</td>
<td>0</td>
<td>Perfect free assumption according to specification$^4$</td>
</tr>
</tbody>
</table>

#### 5.5.1 General results

Since the frictional force of movable bearing acts along LG direction, the analytical results are only compared in LG direction. The response displacement histories on column in LG direction are compared in Figure 5.33.

![Figure 5.33 Comparison of Response Displacement History on Column Top in Longitudinal Direction](image)

As been explained in Section 5.3, the response displacement history in the Case 1 ($\mu=0.12$) coincides with the experimental result well, although both the positive and the negative peaks is slightly overestimated. It can be observed as well in Figure 5.33 that the response displacement in Case 2 ($\mu=0.00$) is generally more violent than both Case 1 and experiment. After 30.0 s, the column still vibrates and is more difficult to settle down in Case 2. Finally, residual displacement of only 0.0012 m is got.

The comparison of the peak displacement and the residual displacement is summarized in Figure 5.34.
5.5.2 Evaluation on residual response

It has been explained in former section that although a safer assessment of the peak displacement can be expected by friction-free assumption, the residual displacement is significantly underestimated. Thus, further evaluation is conducted based on the comparison between Case 1 and Case 2.

The horizontal load histories on column top are compared in Figure 5.35 for the end of analysis (25.0 ~ 40.0 s).

It can be seen that from 25.0 s to 30.0 s, the variation of horizontal load in Case 2 ($\mu=0.00$) is still violent. After 30.0 s, because of the input of 0-acceleration wave, the horizontal load acting on column top becomes almost steady in Case 1 ($\mu=0.12$). However, the variation amplitude of horizontal load in Case 2 decreases slowly, around the X-axis (which means the horizontal load of 0 kN).

It can be inferred that due to the absence of frictional force from movable bearing in Case 2, residual horizontal load acting on column top is approximately zero and can be ignored. As been explained in Section 5.4 in details, under the excitation by E-W component of Takatori wave the residual displacement is mainly caused by the residual horizontal load on column top.
Therefore, with only ignorable residual horizontal load in Case 2, the residual displacement is very small, compared to Case 1 and experiment result.

Figure 5.35 Comparison of Horizontal Load Histories on Column Top in Longitudinal Direction (25.0 ~ 40.0 s)

To sum it up, special attention should be paid that by assuming the frictional force of movable bearing as 0 based on specification [5.4], the residual displacement will be significantly underestimated, because no residual horizontal load on column top occurs.

5.6 Experimental Setup for C1-2

As mentioned in Section 5.1, C1-2 is the specimen that was designed to damage by shear force as a typical column which was built in the 1970s. In this section, the setup of C1-2 will be explained in details.

5.6.1 C1-2 specimen and experiment setup

As shown in Figure 5.36, C1-2 specimen was assumed to damage by shear as a typical column which was built in 1970s. It is a 7.5 m tall, 1.8 m diameter reinforced concrete column. It was designed as a full scale model based on a combination of static lateral force method and working stress design based on 1964 Design Specification of Steel Road Bridges [5.1], with 2 layers of cut-off LG bars. As shown in Figure 5.36, the column has totally 3 layers of LG reinforcing bars with 29 mm diameter, respectively 32, 32 and 16 bars at outer, middle and inner layers. The middle layer was cut off at height of 3.86 m (namely upper cut-off point) and the inner at height of 1.88 m (namely lower cut-off point). Deformed circular hoops with diameter of 13 mm were arranged at 300 mm interval, except outer hoops at top 1.15 m zone and at base 0.95 m zone with 150 mm interval. Stirrups are lap spliced with 390 mm (30 times of its diameter). As a consequence, the LG bar ratio is 2.02% beneath the lower cut-off point, 1.62% beneath
the upper cut-off point, and 0.81% above the upper cut-off point, while the volumetric ratio of hoop is 0.106% for middle and 0.422% for and base. Based on material tests, the average yield strength of LG bars and hoop bars are 383 MPa and 409 MPa, and the elastic modulus of them are 207 GPa and 196 GPa respectively. Therefore, their yield strain can be got as 1850μ and 2050μ. Additionally, the compressive strength of concrete is 30.8 MPa based on material test.

As shown in Figure 5.2 for the general setup of experiment, two simply supported steel decks were set upon the RC column and two steel bents (side bents). Four mass blocks were fixed on decks to simulate the dead load. Between beck and column, 2 fixed bearings (fixed in both longitudinal and transverse directions) and 4 sliders (only provide upward support) were set. The footing of C1-2 specimen was anchored as fix on shake table of E-Defense. The table was excited using E-Takatori ground motion (which was modified from the observed Takatori ground motion by taking the soil-structure interaction based on FEM analysis into consideration) in 3 directions. Main excitation using 100% E-Takatori ground motion was conducted only once, since great failure have occurred to the column.

5.6.2 Design resistance and assumed damage

To assess the failure pattern, the development length \( l_d \), and the shear resistance \( V_R \) is defined

![Diagram](image_url)
as following according to JSCE \[^{[5,15]}\]:

\[
l_d = \frac{\left(\frac{f_y}{1.25\sqrt{f_c}} - 13.3\right)\Phi}{0.318 + 0.795\left(\frac{c}{\Phi} + \frac{1.5A_t}{3\Phi}\right)}
\]  

(5.1)

\[
V_R = V_C + V_S
\]

(5.2)

where,

- \(f_y\) : yield strength of longitudinal reinforcement;
- \(f_c\) : strength of concrete;
- \(c\) : minimum of thickness of cover concrete and half of distance between longitudinal reinforcement;
- \(\Phi\) : diameter of longitudinal reinforcement;
- \(A_t\) : area of reinforcement perpendicular to assumed failure surface;
- \(V_C\) : shear resistance by concrete;
- \(V_S\) : shear resistance by steel reinforcement.

Therefore, the development length for upper cut-off is 440 mm (15.2 \(\Phi\)). Correspondingly, the zone above just point of cut-off, the zone in development length, and the zone below development length, is defined as Section I, II and III respectively (Figure 5.36). Besides, the proposed shear strength by Kawano \[^{[5,16]}\] is applied to concrete. With the flexural resistance, the resistance distribution along height can be drawn in Figure 5.36 (b), and the P-\(\delta\) relation can be drawn in Figure 5.37.
Accordingly, the assumed failure events are listed as:

1. Flexural yield of LG bars at Section II;
2. Shear failure at Section I;
3. Flexural ultimate stage of Section II.

### 5.7 Experimental Response for C1-2

#### 5.7.1 General experimental response

Subjecting to E-Takatori excitation, the response displacement orbit at the top of column (height of 7.5 m) is shown in Figure 5.38. It should be noted that NS and EW directions correspond to the TR and LG directions respectively of the model. It can be found that the column main response in SW-NE direction.

![Figure 5.38 Response Displacement Orbit on Column Top](image)

Some key time points of important damage events are also marked in Figure 5.38 and listed in Table 5.4 (also refer to Figure 5.39 and Figure 5.40).
Table 5.4 General Damage Events

<table>
<thead>
<tr>
<th>Time Point</th>
<th>Response and Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base</td>
</tr>
<tr>
<td>[a] 3.45s</td>
<td>1st yield of LG bar</td>
</tr>
<tr>
<td>[b] 3.55s</td>
<td>1st yield of LG bar</td>
</tr>
<tr>
<td>[c] 4.07s</td>
<td>1st visible horizontal crack + yield of half LG bar</td>
</tr>
<tr>
<td>[d] 4.12s</td>
<td>1st visible horizontal crack + yield of half LG bar</td>
</tr>
<tr>
<td>[e] 4.26s</td>
<td>1st diagonal crack + 1st yield of ties</td>
</tr>
<tr>
<td>[f] 4.33s</td>
<td>Opposite diagonal crack</td>
</tr>
<tr>
<td>[g] 5.37s</td>
<td>Collision between lateral beam and catching frame</td>
</tr>
<tr>
<td>[h] 6.60s</td>
<td></td>
</tr>
<tr>
<td>[i] 6.87s</td>
<td></td>
</tr>
</tbody>
</table>

The first yield of LG bars occurred at about [a] 3.45 s at base of column, followed by the first yield of LG bars at upper cut-off point at [b] 3.55 s. It can be found from the orbit figure, the displacement at these time points were still not great. Then, a visible horizontal crack was firstly developed from [c] 4.07 s to [d] 4.12 s along NW to E surfaces at height of about 3.9 m (position near the upper cut-off point). After reaching the temporary peak resisting load (1418 kN) at [e] 4.26 s (refer to Figure 5.40), these horizontal cracks developed into diagonal cracks until [f] 4.33 s (also the temporary peak displacement).

Then, the column kept vibrating, resulting in following horizontal cracks possibly due to flexural, and diagonal cracks possibly due to shear. After reaching the following peak displacement at [g] 5.37 s, the spalling-off of the covering concrete began to occur at about 6.04 s at N and NW surfaces near the upper cut-off point, and this developed as well at S and SW surfaces at about [h] 6.60 s. Afterwards, the column continuously responded toward SW direction with spalling off of covering concrete and the bottom of lateral beam at the edge collided with the catch system at about [i] 6.87 s. Because this may influence the response of the column, the discussion in following is mainly conducted until the time point of this collision.
5.7.2 Failure of column based on experimental test

To evaluate the failure, the history of combined displacement is plotted in **Figure 5.39** for the time period from 2.00 s to 6.87 s.

**Figure 5.39 Response History of Combined Lateral Displacement at Top of Column**

It becomes obvious to us that the first shear crack occurred at one peak displacement at 4.33 s, after the flexural crack occurred around 4.07 s. Following peak greater than that at 4.33 s occurred at 5.37 s. Then, great failure occurred at upper cut-off point and column base and collision between the lateral beam and catch system happened.

**Figure 5.40 Response Load-Displacement Hysteresis on Top of Column**
Similarly, the load-displacement (P-δ) hysteresis is illustrated in Figure 5.40. Here, both the displacement and the lateral load are calculated into the combined direction based on measured results in separated longitudinal and transverse direction. Especially, the hysteresis from 4.00 ~ 4.50 s is highlighted.

It can be observed that the column behaved roughly according to the design P-δ (based on flexure), until the occurrence of the first horizontal crack at [c] 4.07 s, although the stiffness is slight smaller after the first yields of LG bars at the base of column (at [a] 3.45 s) and the upper cut-off point (at [b] 3.55 s). In spite of the sudden decrease just after [c] 4.07 s, the lateral load increased again and got to about same value until [d] 4.12 s, at when the horizontal cracks began to develop in diagonal direction.

Then, the lateral load increased gradually to the temporary peak of 1418 kN at [e] 4.26 s. This is actually smaller than the shear resistance of both Section I and II (1613 kN and 2062 kN). However, with further development of the diagonal cracks, the lateral load dropped to 1224 kN and the displacement reached its temporary peak, at [f] 4.33s. This lateral load was about 86% of the former peak value at [e] 4.26 s, suggesting the resistance loss probably due to the noticeable occurrence of horizontal and diagonal cracks. In following peaks (for example [g] 5.37 s), the lateral load could not exceed the peak lateral load at [e] 4.26 s. Furthermore, after [h] 6.60 s, the response displacement became much greater with smaller lateral load, because of great failure such as crushing of concrete at upper cut-off.

Thus, [f] 4.33 s is suitable to be considered as the start point when ultimate stage was reached, since the obvious decrease of lateral load.

5.8 Evaluation on Failure Mechanisms and Shear Resistance

General response of C1-2 in experiment was stated in Section 5.7. In this section, for verifying shear resistance and its mechanisms, the detailed response, as strain of LG bars and hoops, their relationship with crack, and the failure progress, will be evaluated.

5.8.1 Detailed damage and shear failure surface

As been mentioned, LG bars of the middle layer were cut off at the height of 3.86 m, which is called the upper cut-off position. Around this position, the strain of LG bars and hoops was measured at height of 3.90 m, which is actually higher than the just point of cut-off by about 1.4Φ as shown in Figure 5.41.
Blow this, the strain of LG bars and hoops was measured at actual height of 3.60 m (-9.0Φ), 3.30 m (-19.3Φ), and 3.00 m (-29.7Φ), as shown in Figure 5.41. To evaluate different damage at different height around the upper cut-off position, the response histories of average strain in LG bars (in Figure 5.42) and hoops (in Figure 5.43) at 3.90 m (+1.4Φ), 3.60 m (-9.0Φ), 3.30 m (-19.3Φ) and 3.00 m (-29.7Φ) (position shown in Figure 5.41) are plotted and compared. Here, the average strain at each height is defined as the average value of total 8 strain gauges attached on LG bars or hoops at a particular height. Thus, this average strain is suitable to explain the general damage development at each height, rather than the strain of any single strain gauge to explain the local damage condition.
For the average strain history shown in Figure 5.42 (b), it can be found that the value measured at height of 3.90 m (1.4Φ) increases rapidly from [c] 4.07 s (first visible horizontal crack) and exceeds the yield strain of 1850μ, probably due to flexural response. Then, it keeps raising and reaches over 8000μ at [f] 4.33 s (when diagonal cracks firstly occurred). After [g] 5.37 s, the strain at this height becomes negative, suggesting the possible buckling of LG bars may have occurred. However, the strain at height of 3.60 m (-9.0Φ) only slightly exceeds the yield strain once and does not reach to notable value until [g] 5.37 s. Besides, the average strain on the other two heights does not develop significantly. As a result, it can be inferred that the flexural response of column firstly affects the LG bars near the just point of cut-off position, and this influence extends to lower sections with further damage.

On the other hand, as the average strain of hoops illustrated in Figure 5.43, the strain develops significantly at height of 3.60 m (-9.0Φ) from [c] 4.07s to [f] 4.33 s, beyond the yield strain
(2050μ) of hoops. Additionally, the strain at two lower heights, 3.30 m (-19.3Φ) and 3.00 m (-29.7Φ), also reaches yield strain. The diagonal shear cracks occurred at [f] 4.33 s is possibly the dominant reason of this notable response strain at these heights lower than the cut-off position. Then, the average strain in hoops at height of 3.90 m (+1.4Φ) increases noticeably beyond yield and reaches similar level of that at lower heights by [g] 5.37 s. Considering the negative strain of LG bars at this height (seen from Figure 5.42), the possible buckling of LG bars can be considered as the main cause of the damage of hoops.

To further evaluate the failure phenomenon and the response strain, and their interactive relationship, Figure 5.44 is plotted.

Figure 5.44 Development of Damage Event (4.07 ~ 4.33 s, at when shear crack occurred firstly)

Figure 5.44 shows the detailed crack condition, yield of LG bars and hoops, during the time period when horizontal cracks and diagonal cracks occurred. Here, to explain the progress of failure, this time period is separately evaluated for the occurrence of the horizontal cracks (from [c] 4.07 s to [d] 4.12 s, and marked by solid rectangular and thick lines) and for the occurrence of the diagonal cracks (from [d] 4.12 s to [f] 4.33 s, marked by hollow rectangular and the thin
Firstly, for the first period (from [c] 4.07 s to [d] 4.12 s), the cracks occurred in the surfaces of column from E side to NW side at the height of about 3.80 ~ 3.90 m. Besides, 4 of total 8 LG bars (the 1st layer without cut-off) at height of 3.9 m reached yield, as well as another 2 at 4.2 m and only 1 at 3.60 m. Therefore, according to the yield condition of the LG bars, it is reasonable to assume that internal crack may develop until half of the section at 3.90 m. Then, from [d] 4.12 s to [f] 4.33 s, the formerly occurred horizontal cracks extended diagonally downwards from NW side to W side until height of about 3.00 m, and from E side to almost S side. In additional, hoops began to yield at several places, such as 3.60 m and 3.90 m in NW side and 3.00 m and 3.30 m in W side, simultaneously. Thus, based on the measured result of strain gauge on hoops, the internal failure surface can be assumed and the angles to the vertical axis can be got as about 40°, by connecting Point A (3.9 m, NW) and Point B (3.0 m, W) shown in Figure 5.44.

### 5.8.2 Evaluation on shear resistance

Based on the explanation in former section, it can be concluded for C1-2 specimen, the shear failure occurred at the upper cut-off position after the yield of several LG bars and the horizontal cracks until half of section. After this horizontal crack (about 90°) due to flexure, as shown in Figure 5.45 (b), the shear resistance degraded and the diagonal crack (about 40°) occurred. As a result, by connecting the start point and the end point of the actual cracks, the actual shear angle was got as about 60° to the vertical axis. This suggests that the horizontal crack due to flexure caused the larger shear angle of 60°, compared to the standard assumption of 45° shear surface based on specification. This further reduced area of concrete and number of hoops to resist the shear load. Thus, the shear resistance ($V'_R$) by the actual failure surface can be expressed as:

$$V'_R = \tau_y A_c \cot \theta + (f_y A_h / s) d \cot \theta$$

(5.2)

where,

- $\tau_y$ : shear strength by Kawano[6] (=0.76 Mpa, with LG bar ratio of 1.16%);  
- $A_c$ : area of concrete section;  
- $\theta$ : angle between shear crack and vertical axis (refer to Figure 5.45 (a));  
- $A_h$ : area of hoops in shear failure section;  
- $s$ : spacing between hoops;  
- $d$ : height of section.

Therefore, the newly calculated shear resistance based on the actual failure surface can be calculated $V'_{R, II} = 1270$ kN, as plotted in Figure 5.45 (b) for the section below the just point of upper cut-off position (Section II).
As a result, this shear resistance \((V'_{R-II} = 1270 \text{ kN})\), due to reduced area of concrete and number of hoops to resist the shear load, is smaller than the initial shear resistance \((V_{R-II} = 2062 \text{ kN})\) by 38.4% and smaller than the flexural resistance \((F_{y-II} = 1369 \text{ kN})\) by 7.2%. Besides, this shear resistance coincide with the \([f]\) 1224 kN (4.33 s) when ultimate stage was firstly reached, stated in Sub-section 5.7.2 (Figure 5.40).

Figure 5.45 Shear Resistance based on Actual Failure Surface

Figure 5.46 Influence by Shear Angle on Shear Resistance
Besides, the reduction of shear resistance with the increase of shear angle is plotted in Figure 5.46.

It can be found that concrete and hoops provide similar shear resistance and decrease similarly. Therefore, the total shear resistance at decreased from \( V_{R-II} = 2062 \, \text{kN} \) with 45° assumption according to specification to \( V'_{R-II} = 1270 \, \text{kN} \) with 60° in actual failure.

To sum it up, due to the flexural failure at upper cut-off position, horizontal cracks developed until half of section and reduced concrete area and hoop number to resist shear, because of the greater angle (60°) to vertical axis. This caused the range below the upper cut-off (Section II) being the critical section with reduced shear resistance (\( V'_{R-II} = 1270 \, \text{kN} \)) by 38.4%.

5.9 Summary

Based on the dynamic analyses for C1-1 specimen by E-Defense excitation test, the detailed evaluation, especially concentrating on the residual displacement, and the parametric study on frictional coefficient, following conclusions have been drawn:

(1) Based on Case 1, response of column was roughly reappeared, by applying frictional coefficient (0.12) and initial stiffness (104 kN/mm) by element test of movable bearing. Focusing on LG direction, although the response displacement was slightly overestimated (averagely +16.4%), the residual displacement based on analytical result (-1.39 cm) was in good correlation with experimental result (-2.06 cm).

(2) The mechanisms of residual displacement, considering hysteresis behavior and residual loads, were evaluated based on analytical results. The curvature by E-W component (LG direction in analysis) of Takatori wave (-0.398*10^{-3} 1/m) and by vertical load considering \( P-\delta \) effect (-0.338*10^{-3} 1/m) only contributed slightly (17.7% and 15.1%) to the total residual curvature (-2.243*10^{-3} 1/m) at column base. However, due to frictional resistance from movable bearing, the residual horizontal load (13.5 kN) acted on column top. This force led to great curvature at base (-1.507*10^{-3} 1/m) which was 67.2% to the total. With triangle curvature distribution, this notable residual curvature at base resulted in notable residual displacement on column top. Therefore, by taking the friction of movable bearing into account, the residual horizontal load on column, which was transmitted by deck from the frictional resistance of movable bearing on side bents, had great influence on the significant residual displacement.

(3) Based on discussion of friction-free assumption according to specification, residual displacement (-0.12 cm) was got, which was about only 1/12 of -1.39 cm in Case 1, or 1/17 of -2.06 cm in experiment. It was thus concluded that although a safer assessment of the peak displacement could be expected by friction-free assumption, the residual
displacement was significantly underestimated. This was resulted by the absence of residual horizontal load on column top, in Case 2 (μ=0.00) without frictional force from movable bearing.

Based on the experimental results of the RC column with cut-off LG bars based on E-Defense excitation, the detailed response displacement, strain of LG bars and hoops, the development of damage, and their interactive relationship were discussed in details. Following conclusions have been drawn:

(4) In the experiment of C1-2, cracks firstly occurred horizontally at the upper cut-off point probably due to flexural response, and then extended into diagonal cracks due to shear. This resulted in the lateral resistance drop of the column.

(5) According to the detailed evaluation on the failure position, it was found that, at the just point of the upper cut-off position, the LG bar suffered the most significant and earliest failure, such as flexural yield and even buckling. On the other hand, at lower sections, as the height considering the development length (about 15.2Φ for C1-2), hoops suffered yield but not noticeable failure, after the occurrence of flexural cracks. This indicated the just point of the cut-off position should be considered as the crucial section that led to the further severe failure.

(6) Based on the assessment on the failure mechanisms and the shear resistance, it was found that due to early horizontal crack until half of section by flexure, and following diagonal crack by shear, the actual shear angle (60°) is greater (than assumed 45° in specification), below the just point of upper cut-off point. Thus, with smaller concrete area and less hoops to resist shear load, the total shear resistance was reduced greatly by 38.4% and severe shear failure occurred to this height.

References


Chapter 6 Conclusions

In this study, focusing on the seismic behavior and the failure mechanisms of RC bridges with low ductility, the assessment of actual bridge structure, which was affected in actual earthquake, and the evaluation on the full scale experimental specimen by excitation table tests, were presented in this study on following topics:

Topic 1. The failure mechanisms of RC rigid-frame arch bridge;

Topic 2. The influence by axial load variation on the seismic behavior of RC rigid-frame arch bridge and its failure;

Topic 3. The mechanisms of noticeable residual displacement occurred to the RC column;

Topic 4. The mechanisms of shear failure for RC column with cut-off of longitudinal bars.

In Chapter 3, the detailed damage condition of Xiaoyudong Bridge was summarized and assessed. Then, dynamic analyses for Xiaoyudong Bridge for Span 1 and Span 2, and Span 3 and Span 4 separately were performed, followed by the discussion on failure mechanisms. As a consequence, following conclusions have been drawn:

(1) According to the detailed field investigation, significant failures occurred to A1 and Span 1 probably due to the effect of the surface fault at about 10 m behind the right dyke, including about 34 cm displacement of A1 towards the middle, the collisions and failures of legs, and the drop at middle of Span 1. On the other hand, deck of Span 4 might collide with A2. Span 3

(2) Dynamic analyses for Xiaoyudong Bridge, a typical RC rigid-frame arch bridge, were performed for Span 1 and Span 2, and Span 3 and Span 4 separately. According to the analytical result of Span 1 and Span 2, one joint on girder with arch legs of each span might suffer severe failure even beyond ultimate for (0.97 to 1.25 Φu). However, the bottoms of arch legs and inclined legs might suffer damage beyond yield but did not exceed the ultimate stage. Thus,

(3) On the other hand, Span 3 and Span 4 vibrated more extensively due to more movable P3. The maximum response displacement at middle of Span 4 was greater than that of Span 1 by 90% and 33% respectively in horizontal and vertical direction. Besides, severe damage occurred to all girder joints with arch leg, bottom of inclined legs and arch legs, leading to the loss of entire stability. As a consequence, arch leg and inclined leg were not able to
support the girder, and both spans collapsed into the river. In actual damage of AL-4-R, concrete at base 50 cm crashed, and main bars buckled but not broke off. Although damage in analysis at bottom of arch leg was slightly less severe than actual, capacity loss was well reappeared.

In Chapter 4, the influence due to M-N interaction was discussed by performing the nonlinear dynamic analysis with or without the consideration of the M-N interaction for the main supporting member, arch legs, by applying the collapsed Span 3 and Span 4 as representative. As a consequence, following conclusions have been drawn:

1) Based on evaluation on influence of M-N interaction, it was found that by neglecting M-N interaction, flexural response was underestimated. Maximum ultimate ratio was greater by considering M-N interaction, no matter the first yield occurred under an axial load greater or smaller, compared to the axial load by only dead load. Average maximum ultimate ratio of 4 points (1.521) was greater if considering M-N interaction, by about 38% than neglecting that (0.950). Especially when yield occurred early due to smaller axial load than that under only dead load, maximum ultimate ratio (2.237) in case with M-N interaction would be as 2.9 times great as that in case without M-N interaction (0.767). Thus, subjected to extremely high axial load (about 65% of axial capacity), arch legs on Span 3 & 4 suffered extensive failure, indicating that the arch legs, as the main supporting member, had too small sectional area and too few confining reinforcement.

2) For future design of this bridge type, the RC rigid-frame arch bridge, it is possible to apply large sectional area, or to apply more confining reinforcement. This design measures can significantly improve the ductility of the arch leg. By doing so, the severe failure at bottom of arch leg, the key member of this bridge, should have been possibly avoided by adding the ties volume or by enlarging the sectional area.

In Chapter 5, based on the dynamic analyses for C1-1 specimen (a full scale RC column failed in flexural failure) by E-Defense excitation test, the detailed evaluation, especially concentrating on the residual displacement, and the parametric study on frictional coefficient, following conclusions have been drawn:

1) Based on Case 1, response of column was roughly reappeared, by applying frictional coefficient (0.12) and initial stiffness (104 kN/mm) by element test of movable bearing. Focusing on LG direction, although the response displacement was slightly overestimated (averagely +16.4%), the residual displacement based on analytical result (-1.39 cm) was in good correlation with experimental result (-2.06 cm).

2) The mechanisms of residual displacement, considering hysteresis behavior and residual loads, were evaluated based on analytical results. The curvature by E-W component (LG direction in analysis) of Takatori wave (-0.398*10^{-3} 1/m) and by vertical load considering
P-δ effect (-0338*10^{-3} 1/m) only contributed slightly (17.7% and 15.1%) to the total residual curvature (-2.243*10^{-3} 1/m) at column base. However, due to frictional resistance from movable bearing, the residual horizontal load (13.5 kN) acted on column top. This force led to great curvature at base (-1.507*10^{-3} 1/m) which was 67.2% to the total. With triangle curvature distribution, this notable residual curvature at base resulted in notable residual displacement on column top. Therefore, by taking the friction of movable bearing into account, the residual horizontal load on column, which was transmitted by deck from the frictional resistance of movable bearing on side bents, had great influence on the significant residual displacement.

(3) Based on discussion of friction-free assumption according to specification, residual displacement (-0.12 cm) was got, which was about only 1/12 of -1.39 cm in Case 1, or 1/17 of -2.06 cm in experiment. It was thus concluded that although a safer assessment of the peak displacement could be expected by friction-free assumption, the residual displacement was significantly underestimated. This was resulted by the absence of residual horizontal load on column top, in Case 2 (μ=0.00) without frictional force from movable bearing.

In Chapter 5 as well, based on the experimental results of the RC column with cut-off LG bars based on E-Defense excitation, the detailed response displacement, strain of LG bars and hoops, the development of damage, and their interactive relationship were discussed in details. Following conclusions have been drawn:

(4) In the experiment of C1-2, cracks firstly occurred horizontally at the upper cut-off point probably due to flexural response, and then extended into diagonal cracks due to shear. This resulted in the lateral resistance drop of the column.

(5) According to the detailed evaluation on the failure position, it was found that, at the just point of the upper cut-off position, the LG bar suffered the most significant and earliest failure, such as flexural yield and even buckling. On the other hand, at lower sections, as the height considering the development length (about 15.2Φ for C1-2), hoops suffered yield but not noticeable failure, after the occurrence of flexural cracks. This indicated the just point of the cut-off position should be considered as the crucial section that led to the further severe failure.

(6) Based on the assessment on the failure mechanisms and the shear resistance, it was found that due to early horizontal crack until half of section by flexure, and following diagonal crack by shear, the actual shear angle (60°) is greater (than assumed 45° in specification), below the just point of upper cut-off point. Thus, with smaller concrete area and less hoops to resist shear load, the total shear resistance was reduced greatly by 38.4% and severe shear failure occurred to this height.
Based on aforementioned evaluations in this study, which concentrates on the full scale reinforced concrete structures, presented the new evaluations on the failure mechanisms and on the influence due to the variation of axial load for RC arch bridges, and presented the original research on the mechanisms of residual displacement for RC bridge columns.
LIST OF PUBLICATIONS

Papers in Peer-Reviewed Journals

Chapter 3


Chapter 4


Chapter 5


[2] Shi Z., Kosa K. and Sasaki T., “Evaluation on Shear Failure Mechanisms of a RC Column with Cut-off Bars based on E-Defense Excitation”, Proceedings of the Japan Concrete Institute, Japan Concrete Institute, Vol. 37, 6 pages, (received)


Papers in Conference Proceedings

Chapter 3


Performance-based Seismic Design Method for Bridges, Japan Society of Civil Engineers, pp.149-156, Tokyo, Japan, Jul. 2011


Chapter 4


Chapter 5


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