Lampiran 1 Data Uji Tarik Baja

Kurva Hasil Uji Tarik (bagian badan)

Kurva Tegangan vs Regangan Hasil Uji Tarik Baja (bagian sayap)
Lampiran 2 Beban vs Regangan Benda Uji 1

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Kurva Beban vs Regangan Strain Gauge Rossete 5,6,7

Kurva Beban vs Regangan Strain Gauge Rossete 8,9,10

Kurva Beban vs Regangan Strain Gauge Rossete 11,12,13
Lampiran 3: Beban vs Regangan Benda Uji 2

Kurva Beban vs Regangan
Strain Gauge 1

Kurva Beban vs Regangan
Strain Gauge 2

Kurva Beban vs Regangan
Strain Gauge 3

Kurva Beban vs Regangan
Strain Gauge 4
15. ECCENTRICALLY BRACED FRAMES (EBF)

15.1. Scope
Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the links when subjected to the forces resulting from the motions of the design earthquake. The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened links, except where permitted in this Section. In buildings exceeding five stories in height, the upper story of an EBF system is permitted to be designed as an OCBF or a SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the applicable building code. EBF shall meet the requirements in this Section.

15.2. Links

15.2a. Limitations
Links shall meet the requirements of Section 8.2b.

The web of a link shall be single thickness. Double-plate reinforcement and web penetrations are not permitted.

15.2b. Shear Strength
Except as limited below, the link design shear strength, \( V_r \), and the allowable shear strength, \( \frac{V_r}{\Omega} \), according to the limit state of shear yielding shall be determined as follows:

\[
\begin{align*}
V_r & = \text{nomininal shear strength of the link, equal to the lesser of } V_r = 2M_y f_y, \text{ kips (N)} \\
\phi & = 0.90 \text{ (LRFD)} \\
\Omega & = 1.67 \text{ (ASD)} \\
\end{align*}
\]

where

\[
\begin{align*}
M_y & = F_y Z, \text{ kip-in. (N-mm)} \\
V_r & = 0.6F_y A_n, \text{ kips (N)} \\
e & = \text{link length, in. (mm)} \\
A_n & = (d-2c)n \\
\end{align*}
\]

The effect of axial force on the link available shear strength need not be considered if

\[
P_a \leq 0.15P_r \text{ (LRFD)}
\]

or

\[
P_a \leq (0.15/1.5)P_r \text{ (ASD), as appropriate.}
\]

where

\[
\begin{align*}
P_a & = \text{required axial strength using LRFD load combinations, kips (N)} \\
P_r & = \text{required axial strength using ASD load combinations, kips (N)} \\
P_y & = \text{nominal axial yield strength } = F_y A_n, \text{ kips (N)}
\end{align*}
\]

Seismic Provisions for Structural Steel Buildings, March 9, 2005, incl. Supplement No. 1
If \( P_{e} > 0.15P_{l} \) (LRFD)

or

\( P_{e} > (0.15/1.5)P_{l} \) (ASD), as appropriate, the following additional requirements shall be met:

1. The available shear strength of the link shall be the lesser of

\[ V_{p} = V_{p} \sqrt{1 - \left( \frac{P_{l}}{P_{u}} \right)^{2}} \quad \text{or} \quad \frac{V_{p}}{\Omega_{e}} \text{ and } 2\left( \frac{M_{p}}{\Theta_{e}} \right)/\Omega_{e} \text{ (ASD), as appropriate,} \]

where

\[ \psi_{v} = 0.90 \text{ (LRFD)} \quad \Omega_{e} = 1.67 \text{ (ASD)} \]

\[ M_{p} = 1.18M_{p} \left[ 1 - \left( \frac{P_{l}}{P_{u}} \right)^{2} \right] \]

\[ P_{e} = P_{l} \text{ (LRFD)} \text{ or } P_{l} \text{ (ASD), as appropriate} \]

\[ P_{e} = P_{l} \text{ (LRFD)} \text{ or } P_{l}/1.5 \text{ (ASD), as appropriate} \]

2. The length of the link shall not exceed:

(a) \( [1.15 - 0.5\rho'(A_{w}/A_{p})]1.6M_{p}/V_{p} \) when \( \rho' \geq 0.3 \)

(b) \( 1.6M_{p}/V_{p} \) when \( \rho'(A_{w}/A_{p}) < 0.3 \)

where

\[ A_{w} = (d - 2t_{w}) \]

\[ \rho' = P_{l}/V_{l} \]

and where

\[ V_{l} = V_{l} \text{ (LRFD)} \text{ or } V_{l} \text{ (ASD), as appropriate} \]

\[ V_{s} = \text{required shear strength based on LRFD load combinations, kips} \]

15.2c. Link Rotation Angle

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift, \( \Delta \). The link rotation angle shall not exceed the following values:

(a) 0.08 radians for links of length \( 1.6M_{p}/V_{p} \) or less.

(b) 0.02 radians for links of length \( 2.6M_{p}/V_{p} \) or greater.

(c) The value determined by linear interpolation between the above values for links of length between \( 1.6M_{p}/V_{p} \) and \( 2.6M_{p}/V_{p} \).

15.3. Link Stiffeners

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than \( (b_{f} - 2t_{w}) \) and a thickness not less than \( 0.75t_{w} \) or \( \frac{1}{8} \) in. (10 mm).
whichever is larger, where \( b_f \) and \( t_w \) are the link flange width and link web thickness, respectively.

Links shall be provided with intermediate web stiffeners as follows:

(a) Links of lengths \( \sqrt{4M_p/V_p} \) or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding \((3\pi - d/5)\) for a link rotation angle of 0.08 radian or \((5\pi - d/5)\) for link rotation angles of 0.02 radian or less. Linear interpolation shall be used for values between 0.08 and 0.02 radian.

(b) Links of length greater than \( 2.6M_p/V_p \) and less than \( 5M_p/V_p \) shall be provided with intermediate web stiffeners placed at a distance of 1.5 times \( b_f \) from each end of the link.

(c) Links of length between \( 1.6M_p/V_p \) and \( 2.6M_p/V_p \) shall be provided with intermediate web stiffeners meeting the requirements of (a) and (b) above.

(d) Intermediate web stiffeners are not required in links of lengths greater than \( 5M_p/V_p \).

(e) Intermediate web stiffeners shall be full depth. For links that are less than 25 in. (635 mm) in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than \( t_w \) or \( 3/4 \) in. (10 mm), whichever is larger, and the width shall not be less than \( (b_f/2) - t_w \). For links that are 25 in. (635 mm) in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The required strength of fillet welds connecting a link stiffener to the link web is \( A_f F_y \) (LRFD) or \( A_f F_y / \sqrt{2} \) (ASD), as appropriate, where \( A_f \) is the area of the stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is \( A_f F_y / A \) (LRFD) or \( A_f F_y / A(1.5) \) (ASD).

15.4. Link-to-Column Connections

Link-to-column connections must be capable of sustaining the maximum link rotation angle based on the length of the link, as specified in Section 15.2c. The strength of the connection measured at the column face shall equal at least the nominal shear strength of the link, \( V_L \), as specified in Section 15.2b at the maximum link rotation angle.

Link-to-column connections shall satisfy the above requirements by one of the following:

(a) Use a connection prequalified for EBF in accordance with Appendix P.

(b) Provide qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in research literature or documented tests performed for other projects that are representative of project conditions, within the limits specified in Appendix S.
(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

Exception: Where reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length, the link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such links are used and the link length does not exceed $1.6M_p/V_p$, cyclic testing of the reinforced connection is not required if the available strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon the strain-hardened link as described in Section 15.6. Full depth stiffeners as required in Section 15.3 shall be placed at the link-to-reinforcement interface.

15.5. Lateral Bracing of Link

Lateral bracing shall be provided at both the top and bottom link flanges at the ends of the link. The required strength of each lateral brace at the ends of the link shall be $P_b = 0.06 M/L h_n$, where $h_n$ is the distance between flange centroids in in. (mm).

For design according to Specification Section B3.3 (LRFD)

$$M_r = M_{set} = R_Z F_c$$

For design according to Specification Section B3.4 (ASD)

$$M_r = M_{set}/1.5$$

The required brace stiffness shall meet the provisions of Equation A-6-8 of the Specification, where $M_r$ is defined above, $C_2 = 1$, and $L_b$ is the link length.

15.6. Diagonal Brace and Beam Outside of Link

15.6a. Diagonal Brace

The required combined axial and flexural strength of the diagonal brace shall be determined based on load combinations stipulated by the applicable building code. For load combinations including seismic effects, a load $Q_1$ shall be substituted for the term $E$, where $Q_1$ is defined as the axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link $R V_n$ where $V_n$ is as defined in Section 15.2b. The available strength of the diagonal brace shall comply with Specification Chapter H.

Brace members shall meet the requirements of Section 8.2a.

15.6b. Beam Outside Link

The required combined axial and flexural strength of the beam outside of the link shall be determined based on load combinations stipulated by the applicable building code. For load combinations including seismic effects, a load $Q_1$ shall be substituted for the term $E$ where $Q_1$ is defined as the forces generated by at least 1.1 times the expected nominal shear strength of the link $R V_n$ where $V_n$
is as defined in Section 15.2b. The available strength of the beam outside of the link shall be determined by the Specification, multiplied by $R_r$.

User Note: The diagonal brace and beam segment outside of the link are intended to remain essentially elastic under the forces generated by the fully yielded and strain hardened link. Both the diagonal brace and beam segment outside of the link are typically subjected to a combination of large axial force and bending moment, and therefore should be treated as beam-columns in design, where the available strength is defined by Chapter H of the Specification.

At the connection between the diagonal brace and the beam at the link end of the brace, the intersection of the brace and beam centerlines shall be at the end of the link or in the link.

15.6c. Bracing Connections

The required strength of the diagonal brace connections, at both ends of the brace, shall be at least equal to the required strength of the diagonal brace, as defined in Section 15.6a. The diagonal brace connections shall also satisfy the requirements of Section 13.3c.

No part of the diagonal brace connection at the link end of the brace shall extend over the link length. If the brace is designed to resist a portion of the link end moment, then the diagonal brace connection at the link end of the brace shall be designed as a fully-restrained moment connection.

15.7. Beam-to-Column Connections

If the EBF system factors in the applicable building code require moment resisting connections away from the link, then the beam-to-column connections away from the link shall meet the requirements for beam-to-column connections for OMF specified in Sections 11.2 and 11.5.

If the EBF system factors in the applicable building code do not require moment resisting connections away from the link, then the beam-to-column connections away from the link are permitted to be designed as pinned in the plane of the web.

15.8. Required Strength of Columns

In addition to the requirements in Section 8.3, the required strength of columns shall be determined from load combinations as stipulated by the applicable building code, except that the seismic load $E$ shall be the forces generated by 1.1 times the expected nominal shear strength of all links above the level under consideration. The expected nominal shear strength of a link is $R_sV_s$ where $V_s$ is as defined in Section 15.2b.

Column members shall meet the requirements of Section 8.2b.

15.9. Protected Zone

Links in EBFs are a protected zone, and shall satisfy the requirements of Section 7.4. Welding on links is permitted for attachment of link stiffeners, as required in Section 15.3.
15.10. Demand Critical Welds

Complete-joint-penetration groove welds attaching the link flanges and the link web to the column are demand critical welds, and shall satisfy the requirements of Section 7.3b.

16. BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

16.1. Scope

Buckling-restrained braced frames (BRBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. BRBF shall meet the requirements in this Section. Where the applicable building code does not contain design coefficients for BRBF, the provisions of Appendix R shall apply.

16.2. Bracing Members

Bracing members shall be composed of a structural steel core and a system that restrains the steel core from buckling.

16.2a. Steel Core

The steel core shall be designed to resist the entire axial force in the brace.

The brace design axial strength, \( P_{wc} \) (LRFD), and the brace allowable axial strength, \( P_{wc} \Omega \) (ASD), in tension and compression, according to the limit state of yielding, shall be determined as follows:

\[
P_{wc} = F_{wc} A_{wc}
\]

\[
\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}
\]

where

\( F_{wc} \) = specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, ksi (MPa)

\( A_{wc} \) = net area of steel core, in.\(^2\) (mm\(^2\))

Plates used in the steel core that are 2 in. (50 mm) thick or greater shall satisfy the minimum notch toughness requirements of Section 6.3.

Splices in the steel core are not permitted.

16.2b. Buckling-Restraining System

The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.

The buckling-restraining system shall limit local and overall buckling of the steel core for deformations corresponding to 2.0 times the design story drift. The buckling-restraining system shall not be permitted to buckle within deformations corresponding to 2.0 times the design story drift.

User Note: Conformance to this provision is demonstrated by means of testing as described in Section 16.2c.
S6. LOADING HISTORY

S6.1. General Requirements
The test specimen shall be subjected to cyclic loads according to the requirements prescribed in Section S6.2 for beam-to-column moment connections in special and intermediate moment frames, and according to the requirements prescribed in Section S6.3 for link-to-column connections in eccentrically braced frames.

Loading sequences other than those specified in Sections S6.2 and S6.3 may be used when they are demonstrated to be of equivalent or greater severity.

S6.2. Loading Sequence for Beam-to-Column Moment Connections
Qualifying cyclic tests of beam-to-column moment connections in special and intermediate moment frames shall be conducted by controlling the interstory drift angle, θ, imposed on the test specimen, as specified below:

1. 6 cycles at \( \theta = 0.00375 \) rad
2. 6 cycles at \( \theta = 0.005 \) rad
3. 6 cycles at \( \theta = 0.0075 \) rad
4. 4 cycles at \( \theta = 0.01 \) rad
5. 2 cycles at \( \theta = 0.015 \) rad
6. 2 cycles at \( \theta = 0.02 \) rad
7. 2 cycles at \( \theta = 0.03 \) rad
8. 2 cycles at \( \theta = 0.04 \) rad

Continue loading at increments of \( \theta = 0.01 \) radian, with two cycles of loading at each step.

Seismic Provisions for Structural Steel Buildings, March 9, 2005, incl. Supplement No. 1
American Institute of Steel Construction, Inc.
S6.3. Loading Sequence for Link-to-Column Connections

Qualifying cyclic tests of link-to-column moment connections in eccentrically braced frames shall be conducted by controlling the total link rotation angle, $\gamma_{\text{tot}}$, imposed on the test specimen, as follows:

1. 6 cycles at $\gamma_{\text{tot}} = 0.0075$ rad
2. 6 cycles at $\gamma_{\text{tot}} = 0.05$ rad
3. 6 cycles at $\gamma_{\text{tot}} = 0.075$ rad
4. 6 cycles at $\gamma_{\text{tot}} = 0.01$ rad
5. 4 cycles at $\gamma_{\text{tot}} = 0.015$ rad
6. 4 cycles at $\gamma_{\text{tot}} = 0.02$ rad
7. 2 cycles at $\gamma_{\text{tot}} = 0.03$ rad
8. 1 cycle at $\gamma_{\text{tot}} = 0.04$ rad
9. 1 cycle at $\gamma_{\text{tot}} = 0.05$ rad
10. 1 cycle at $\gamma_{\text{tot}} = 0.07$ rad
11. 1 cycle at $\gamma_{\text{tot}} = 0.09$ rad

Continue loading at increments of $\gamma_{\text{tot}} = 0.02$ radian, with one cycle of loading at each step.

S7. INSTRUMENTATION

Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Section S9.

S8. MATERIALS TESTING REQUIREMENTS

S8.1. Tension Testing Requirements for Structural Steel

Tension testing shall be conducted on samples of steel taken from the material adjacent to each test specimen. Tension-test results from certified mill test reports shall be reported but are not permitted to be used in place of specimen testing for the purposes of this Section. Tension-test results shall be based upon testing that is conducted in accordance with Section S8.2. Tension testing shall be conducted and reported for the following portions of the test specimen:

1. Flange(s) and web(s) of beams and columns at standard locations
2. Any element of the connection that supplies inelastic rotation by yielding

S8.2. Methods of Tension Testing for Structural Steel

Tension testing shall be conducted in accordance with ASTM A6/A6M, ASTM A370, and ASTM E8, with the following exceptions:

1. The yield stress, $F_y$, that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method at 0.002 strain.