Seismic Performances of High Rise R/C Frame Structures Reinforced with High Strength Rebars

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Abstract — Construction of high rise buildings as supporting infrastructures for economic growth has increased significantly in numbers in many big cities around the world. In Indonesia, most of the high-rise buildings constructed are made of reinforced concrete structures. In principles, the use of high-strength concrete, coupled with high strength rebars for high rise r/c buildings will result in more efficient and more constructible r/c constructions. However, in Indonesia, the use of high strength rebars for seismic-resistant r/c buildings is still prohibited. SNI 2847:2013 Section 21 specifies that the yield strength for reinforcing bars used in structural elements of special moment resisting frames is limited to 420 MPa. This provision is meant to limit higher shear and higher bond demand in the structural elements assigned to dissipate seismic energy. This paper presents a study on the use of high strength rebars in seismic resistant r/c buildings. In the study, 20 story buildings located in a region with high seismicity are designed. Two types of rebars are used, i.e., those with the yield strength of 550 MPa and of 690 MPa. The building structures are designed as the special moment resisting frame. The seismic performances of the buildings are then investigated by performing non-linear time history analysis. Seven pairs of scaled ground motions are used for the analysis. From this analysis, the failure mechanism of r/c buildings reinforced with 550 MPa yield strength is still prohibited. SNI 2847:2013 Section 21 specifies that the yield strength for reinforcing bars used in structural elements of special moment resisting frames is limited to 420 MPa. This provision is meant to limit higher shear and higher bond demand in the structural elements assigned to dissipate seismic energy.

Deformed non-prestressed longitudinal reinforcement resisting earthquake-induced moment, axial force, or both, in special moment frames, special structural walls, and all components of special structural walls shall be in accordance with [9]:

1. Actual yield strength based on mill test does not exceed $f_{y}$ by more than 125MPa. (Overstrength Ratio Factor).

2. The ratio of actual tensile strength to the actual yield strength is at least 1.25 (Ultimate Strength Ratio Factor).

If the parameters of rebar do not exceed from that provision, it means that the rebar has enough ductility to be

Keywords — ductility; non-linear time history analysis; performance-based design; high strength rebars

I. INTRODUCTION

The increase of quality of a product will always develop according to the needs and interest of modern era. The mobility of development in the technology of construction material gives an opportunity to provide more effective and efficient infrastructure to answer the human activity that is getting complex. One of measurement of the developing infrastructure is the ability in bearing heavier workload. This is in alignment with the innovation of construction material power that is being used.

The needs for a higher strength of concrete and steel reinforcement bar materials will be the main challenge in the development of the study in the construction of material technology. In principles, the use of high-strength concrete, coupled with high strength rebars for high rise r/c buildings will result in more efficient and more constructive r/c constructions.

However, in Indonesia, the use of high strength rebars for seismic-resistant r/c buildings is still prohibited. SNI 2847:2013 Section 21 specifies that the yield strength for reinforcing bars used in structural elements of special moment resisting frames is limited to 420 MPa. This provision is meant to limit higher shear and higher bond demand in the structural elements assigned to dissipate seismic energy.

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If the parameters of rebar do not exceed from that provision, it means that the rebar has enough ductility to be
used in dissipation energy caused by the cyclic load of the high-rise r/c building.

The security and performance level of the high-rise building structure in receiving applied load is essential considering of the location that is among crowded civilians. This study will elaborate the impact of using grade 550MPa and 690MPa steel reinforcement bar in the progress and seismic performance of high-rise r/c building through Performance Base Design Analysis.

II. MATERIAL AND METHOD

The characteristic of concrete and rebar that are used in seismic high rise r/c building will highly affect the yield mechanism of r/c frame structure produced [1]. The parameters of rebar have to be in the provision of rules to prevent the failure of structure in inelastic phase caused by cyclic loading effect.

The rebar material in reinforcement concrete structure has a major role towards the yield mechanism of moment resistant frame structure. Ductility of rebars that is used has full responsibility in bearing force produced by reduction of base shear in the progress of design, especially for reinforced concrete structure building that designed with special rebar details. Some of the parameters in mechanical properties of rebar that affect plastic deformation of earthquake resistant r/c element structure [1], among others:

- The surface of the rebar (plain or thread) → affects bond strength between concrete and steel reinforcement bars materials.
- The tensile strength of rebar → influence the bond behavior that produced by r/c element structure in inelastic condition.
- Overstrength ratio factor of rebar → that can affect yield mechanism hierarchy of r/c structural building from post-elastic condition through the limits of the inelastic range.
- Ultimate Strength factor of rebar → affects the curvature ductility capacity of reinforced concrete sections.
- Total Elongation → influences the deviations and effective plastic hinges length of r/c structure elements.

The parameters of rebar mechanic behavior depend on the process of the product, starting from the method, composition of chemical elements that are used as the material, up to the goal of the rebar strength. In general, the high strength rebar material has lower ductility capacity than rebar with normal strength. The challenge of development in manufacturing rebar is producing high strength rebar with enough ductility for seismic-resistant reinforced concrete structure.

Commentary ACI 318-14, Section 20.2.2.4, said that for deformed reinforcement in special moment frames and special structural walls, the use of longitudinal reinforcement with strength substantially higher than assumed in Table 20.2.2.4a, ACI 318-14, will lead to higher shear and bond stresses at the time of development of yield moments. In that table, grade 550MPa steel reinforcement bar has to be an optional reinforcement that can be used for flexure, axial and lateral support for the special seismic system [9].

Fig. 1 Actual stress-strain curve for representative samples of various type and grades of ASTM steel reinforcing bars (WJE 2008)

Some ASTM (American Society for Testing and Materials) standard specification which set about the tensile requirements of reinforcement concrete bars to resist seismic load, these are ASTM A615/A615M, ASTM A706/A706M, dan ASTM A1035/A1035M. The various stress-strain relations curve of ASTM steel reinforcement bars is seen in Fig. 1.

A. Material Properties

The project of national research in Japan with the title development of Advanced R/C Building using High Strength Concrete and Reinforcement (new RC Project) has developed design criteria concrete material with compressive strength 30 – 120MPa and rebar material with yield strength 420 – 1200MPa as the construction material for high building earthquake resistant [6]. The achievement of a grade of concrete rebar that is the target research of New RC Project is divided into 4 zones, as shown in Fig. 2.

Fig. 2 Strength of materials and zone for New RC project research and development (Aoyama 2001)

Determining the strength concrete and rebar in this study is based on the classification of zone 1 from the New RC Project research. In using rebar with the strength of 400 – 700 MPa must be followed with concrete of 30 – 60 MPa. The application of high strength rebar in reinforcement concrete element that receives enough axial-cyclic load must be accompanied with high strength concrete which also compatible to hinder slip failure on the surface of bond strength between concrete and rebars.

1) Concrete Material: The compressive strength of concrete used in this study is 35MPa for horizontal structure
element and 45MPa for the vertical. The ratio of horizontal and vertical compressive strength of concrete material should not be more than 1.4 (45MPa/35MPa = 1.286).

Stress-strain curve model of concrete material for confined and unconfined is used in the formula in Kent and Park approach method [13] [20].

2) ASTM A706 Grade 550MPa Reinforcement: ASTM A706 steel is a deformed and plain low-alloy steel bar which has a well-controlled strength of the reinforcing steel. It is shown in the presence of specific requirements regarding not only minimum but also maximum yield stress as well as sustain larger elongations and meet specific chemical composition requirements [3]. ASTM A706 steel is routinely specified, at the minimum, for members expected to form plastic hinges.

The vast majority, approximately 98% of the ASTM A706 straight reinforcing bar actual stress-strain curves that were reviewed for this study have stress-strain relationships that include a linear-elastic portion with a well-defined or sharp yield point, followed by a yield plateau that eventually transitions to strain hardening (EPSH behavior) [3].

Specification ASTM A706 grade 550MPa reinforcement bar that is used in the study analysis are these:

- Yield strength \( f_y \) : 550MPa
- Ultimate strength \( f_u \) : 690MPa
- Modulus of Elasticity \( E_s \) : 200000MPa
- Total elongation \( \varepsilon_{tot} \) : 10 – 12%

The result data for material properties of ASTM A706 grade 550MPa reinforcement bar is taken from the experiment by Drit Sokoli as seen in Table 1.

The experiment shows that overstrength ratios factor of rebars are \( \leq 1.3 \) and ultimate strength ratios factor of rebars are \( \geq 1.25 \).

The average of ASTM A706 grade 550MPa overstrength ratio factor from the experiment is in between of 1 – 1.046 \( \leq 1.25 \). Thus, the overstrength ratio in the calculation of capacity design and the flexural strength of special column detail with this rebar still used 1.25. Total elongation from each specimen is also shown the result between 10 to 12%.

\[ \varepsilon_y \leq \varepsilon_s \leq \varepsilon_{sh} \quad \text{and} \quad f_s = f_y \quad (2) \]

\[ f_s = f_u - (f_u - f_y) \left[ \frac{\varepsilon_{tot} - \varepsilon_s}{\varepsilon_{tot} - \varepsilon_{sh}} \right]^2 \quad (psi) \quad (3) \]

where is an ultimate tensile strain and \( \varepsilon_{sh} \) is a strain of materials before the strain hardening happened. The recommendation value of both parameters to get ASTM A706 grade 550MPa EPSH curve are continuously 0.0954 dan 0.0074 [3]. For total elongation \( \varepsilon_{tot} \) is taken according to specification, that is 12%.

The cyclic test of rebar ASTM A706 has been done in the R/C column structure by Drit Sokoli, B.E. (2014), University of Texas, Austin. The specimens being used are CS60 and CS80, where both specimens are R/C column with identical flexure and shear strength. CS60 used the ASTM706 with grade 420MPa, and CS80 used ASTM A706 with grade 550MPa, both for the longitudinal and transversal specimens. Fig. 4 shows that the specimen of R/C column CS60 and CS80 has identical hysteretic curve and degradation characteristic. Both of the specimens have a stable cyclic performance with two amplitude cyclic load until it reaches drift ratio > 5.5%.

The maximum drift ratio number produced by both specimens exceed the minimum performance objective for the area of collapse prevention in the level of Maximum Considered Earthquake (MCE) Hazard, which is 4% [17].

For \( \varepsilon_s < \varepsilon_y \), \( f_s = E_s \varepsilon_s \quad \text{and} \quad f_s = f_u - (f_u - f_y) \left[ \frac{\varepsilon_{tot} - \varepsilon_s}{\varepsilon_{tot} - \varepsilon_{sh}} \right]^2 \quad \text{(psi)} \quad \text{(3)} \)

3) ASTM A1035 Grade 690MPa Reinforcement: ASTM A1035 steel is a type of steel which has low carbon level (0.15%) and chrome 8 – 11%. The high level of chrome in the contents caused the ASTM A1035 more resistant to corrosion compared to another type of steel. American concrete institute’s innovation task group 6 (ACI ITG-6) through Wiss, Janney, Elstner Associates, Inc. (WJE), conducted laboratory research to obtain the mechanical properties and the characteristic of steel refer to ASTM A1035.

The maximum actual yield strength of steel can be obtained by using 0.2% offset method or extension under load method 0.35%. This is the consequence of high strength steel, where the yield strength does not appear obviously in the stress-strain curve tensile test result.

ACI ITG-6R-10, Design Guide for the use of ASTM A1035/1035M Grade 100 Steel Bars of Structural Concrete (ACI, 2010a), provides the recommendation of designing steps that need to be considered according to the use of high-
strength steel ASTM 1035/1035M in members resisting earthquake effect. The design steps only apply to bars with 690MPa grade.

| Table I | ASTM A706 Re-bars Tension Test Result Grade 80ksi (550MPa) (Drit Sokoll, B. E., University of Texas, 2014) |
|---------|-----------------------------------------------------------------------------------------------------|
| Steel Re-bars Diameter (mm) | Specification of Yield Strength (ksi) | Stress Result | Uniform Elongation (%) | Overstrength Factor (f_y / f_u) | Ultimate Strength Factor |
| 29 (#8) | 80 | 78.8 | 106.2 | 8.735 | 14.1 | 0.985 | 1.348 |
| | | 79.6 | 106.9 | 8.661 | N/A | 0.995 | 1.343 |
| | | 79.1 | 106.8 | 8.992 | 17.6 | 0.680 | 1.350 |
| | | 78.9 | 106.1 | 8.677 | 14.8 | 0.986 | 1.345 |
| Average value | 79.1 | 106.5 | 8.766 | 15.5 | 0.989 | 1.346 |
| 13 (#4) | 80 | 84.3 | 110.9 | 8.705 | 11.8 | 1.054 | 1.316 |
| | | 82.7 | 111.7 | 9.022 | 12.7 | 1.034 | 1.351 |
| | | 84 | 111.8 | 8.825 | 12 | 1.050 | 1.351 |
| Average value | 83.667 | 111.467 | 8.852 | 12.167 | 1.046 | 1.332 |

| Table II | ASTM A1035 Re-bars Tension Test Result Grade 100ksi (690MPa) (WJE, 2008) |
|----------|------------------------------------------------------------------------|
| Steel Re-bars Diameter (mm) | Specification of Yield Strength f_y (ksi) | Stress Corresponding to EUL 0.35% (ksi) | Yield Strength by Office 0.2% Method f_y (ksi) | Tensile Strength f_u (ksi) | Total Elongation ε_u (%) | Overstrength Factor (f_u / f_y) | Ultimate Strength Factor |
| 36 (#12) | 100 | 99 | 129 | 159.7 | - | 1.29 | 1.238 |
| | | 90 | 129 | 159.9 | 9.6 | 1.29 | 1.240 |
| | | 94 | 131 | 159.9 | 1.31 | 1.221 |
| Nilai rata-rata | | 94.3 | 129.7 | 159.8 | 9.1 | 1.297 | 1.233 |
| 25 (#10) | 100 | 86 | 120 | 155.6 | 11 | 1.20 | 1.297 |
| | | 93 | 122 | 155.2 | 10.2 | 1.22 | 1.272 |
| Nilai rata-rata | | 89.500 | 121 | 155.4 | 10.6 | 1.21 | 1.284 |

Specification ASTM A1035 grade 690MPa reinforcement bar that is used in the study analysis are these:

- Yield strength (f_y) : 690MPa
- Tensile strength (f_u) : 1030MPa
- Modulus of elasticity (E) : 200000MPa
- Total elongation (E_total) : 6 – 7%

The result data of ASTM A1035 grade 690MPa reinforcement bar is taken from the experiment by Wiss, Janney, Elstner Associates, Inc. (WJE) as seen in Table 2.

Fig. 5 Illustrating stress-strain RH relationship curve of ASTM A1035 steel Re-bar grade 690MPa

The results of these experiments show that ASTM A1035 grade 690MPa rebar has overstrength ratio factor all of the specimens are ≤ 1.3. However, the ultimate strength ratio of these rebars is ≥ 1.25 only for rebar with diameter cross-section 25mm. The average of ASTM A1035 grade 690MPa overstrength ratio factor that resulted for rebars with diameter cross-section 25mm is ≤ 1.25 so that the overstrength ratio in the calculation of capacity design and the flexural strength of special column detail with this rebar still used 1.25.

From few numbers of overstrength and ultimate strength ratio factor from the specimen in Table 3, grade 690MPa reinforcement bar that is used in structure element that is planned in dissipating seismic energy in this study is with ≤ 25mm diameter rebar. The result of the experiment also shown that total elongation that exceeds the limit of ASTM A1035 grade 690MPa rebars is bigger than 7%.

The approach of stress-strain relationship curve ASTM A1035 grade 690MPa rebar in Fig. 5 obtained by using the method equation of RH curve from Mast’s Equation [14] followed by the limit of rebar tensile strength by ACI ITG-6R-10 [8], where the numbers should not be bigger than the specification of tensile strength. This is equation curve that is used.

For \( \varepsilon_s < 0.0024 \),

\[ f_s = E_s \varepsilon_s \]  \( \text{Equation 4} \)

For \( 0.0024 < \varepsilon_s \leq 0.02 \),


\[ f_s = 1170 - \frac{2.96}{\varepsilon_s + 0.0019} \text{ (MPa)} \]  

(5)

For \( 0.02 < \varepsilon_s \leq \varepsilon_{\text{tot}} \), \( f_s = 1030 \text{ (MPa)} \)  

(6)

For total elongation \( \varepsilon_{\text{tot}} \) is taken according to specification, that is 7%.

**TABLE III**

| No. | Yield Strength Approach Method | Stress \( f_s \) (MPa) | Strain \( \varepsilon_s \) (%) |
|-----|--------------------------------|------------------------|-----------------------------|
| 1   | EUL 0.25%                      | 621.352                | 0.25                        |
| 2   | EUL 0.5%                      | 741.164                | 0.50                        |
| 3   | For\% 0.2%                    | 793.680                | 0.59                        |

For stress average 1 dan 3: 707.856 MPa  
Stress average 1 dan 2: 681.433 MPa

Yield Strength from average measurement: 690.645 MPa

![Fig. 6 Hysteresis response of UC-1.6-10 reinforced concrete column specimen (J. M. Rautenberg; 2011)](image)

From that equation, determination of yield strength rebar through approach method shows that yield strength from rebar stress-strain relations curve of ASTM A1035 grade 690MPa reinforcement bar has identical value with the specification, so the curve is considered applicable to represent the characteristic of mechanical properties of rebar in the analysis process of this study.

The research about the application of ASTM A1035 rebar was conducted by Jeffrey Michael Rautenberg at 2011. UC-1.6-10 and UC-1.6-20 were 2 of 11 of the specimens of R/C column that is using ASTM A1035 120ksi (830MPa). All the parameter of the specimen are kept being similar, except for the axial compression load received by the specimens, each is 0.1 or 0.2 \( f'_c \).

From the hysteresis curve pada Fig. 6 and Fig. 7 it is shown that the specimen of the reinforced concrete column UC-1.6-10 with constant axial load 0.2 \( f'_c \cdot A_g \) has higher energy dissipation than the specimen of reinforced concrete UC-1.6-20 with constant axial load 0.1 \( f'_c \cdot A_g \). Drift ratio capacities that can be reached by both specimens is 5%.

The floor numbers of the building are 20, with two times adjustment of column cross-section properties, there are at 10 and 15 story. The height of the first floor is 5 meters, and the typical floor height is 4 meters.

**B. Design and Detailing Consideration**

The structure of R/C apartment with a typical floor as shown in Fig. 8 is a three dimension typical model used to evaluate the influence of high strength rebar toward the performance of seismic resistant R/C building structure. That structure has a seismic force-resisting system such as special reinforced concrete moment frames (open frame). The system of this building is designed according to some regulation that is valid in Indonesia, which are:

- For seismic load: SNI 1726:2012
- For gravity load: SNI 1727:2013
- R/C Building Structure: SNI 2874:2013

![Fig. 7 Hysteresis Response of UC-1.6-20 Reinforced Concrete Column Specimen (J. M. Rautenberg; 2011)](image)

Fig. 7 Hysteresis Response of UC-1.6-20 Reinforced Concrete Column Specimen (J. M. Rautenberg; 2011)

The application of rebar with bigger than grade 420MPa will affect preliminary of beam and column section properties. For structure element of R/C beam with rebar strength > 420MPa, the estimation of beam height obtained by the multiplication from minimum beam depth arranged in table 9.5(a), SNI 2847:2013, with enlargement factor seen below [22].

![Fig. 8 Typical 3D models of 20 Story R/C apartment building structure](image)
Hiroyuki Aoyama

The enlargement factor of beam depth has a goal to keep the bond strength of concrete to rebars in the flexural element structure sustainable so that it can give an optimum and stable performance.

For structural elements of reinforced concrete columns with special design, SNI 2847:2013 section 21.7.2.3 requires that where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall be at least 20 times the diameter of the largest longitudinal beam bar. The limit of number 20 is the bar diameter column depth ratio which is necessary for beam bar development. If the rebar grade bigger than 420MPa, the comparison between the interior column bar diameter with the column depth follows the equation [6].

$$ h_c = \frac{1}{1.34 \left(1 + \frac{P_u}{A_g f_y}\right) f_y (f_y/c)^{2/3}} d_b \text{ (MPa)} \quad (8) $$

where $h_c$ is column depth, $P_u$ is ultimate axial load, $A_g$ is the cross-sectional area of the concrete material, and $d_b$ is the biggest of longitudinal beam bar diameter that is aligned with the column depth. The cross-sectional area needs to be obtained by the following equation [22].

$$ F_9 / 9 = 6 $$

From Fig. 10, it can be seen that the compressive stress-strain responses of grade 420MPa and grade 550MPa bars closely follow mirrored tensile stress-strain relationship up to a strain of approximately 0.025 for each $s/d_b$ ratio. Critical stress is a stress that happens when the phenomena of longitudinal bar buckling caused when the critical stress started to happen. This fact shows that the limits of spacing transverse reinforcement $6 d_b$ that is regulated in SNI 2874:2013 also applicable in longitudinal reinforcement bar with grade 550MPa.

The compressive stress-strain responses of grade 100ksi (690MPa) with ratio $s/d_b = 6$ shows lower critical strain. 0.025 critical strain achieve by the compressive stress-strain longitudinal bar with the grade over 550MPa, the limit of spacing transverse reinforcement has to be changed to $5 d_b$.

2) Bar Buckling Resistant: SNI 2874:2013 regulation limits the spacing of transverse reinforcement in potential plastic hinges areas of beams and columns and in boundary elements of walls to 6$d_b$ the diameter of the longitudinal bar. This requirement aims to restrain the longitudinal reinforcement and thus delays buckling when the reinforcement undergoes reverse cycles where yielding and hardening occurs in tension and compression in a plastic hinges area.

3) Hysteresis Model Approach: The model of cyclic non-linear response is a method to predict the response characteristic of building structures in bearing the seismic load with various intensity in time history analysis. The goal of this method is to conduct a simulation in holistic deformation movement and deterioration of structure when it experiences post-elastic phase until the collapse caused by the applied load.

From many mechanisms of possible plasticity, the flexure mechanism in concrete reinforced is a plasticity mechanism dominated by the rebar that is being used. Plasticity behavior reinforced concrete will have enough ductility to sustain

$$ P_u = 0.8 \phi [0.85 f_y c \cdot A_g (1 - \rho_s) + f_y A_{st}] \quad (9) $$

$$ A_g = \frac{P_u}{0.85 f_y c (1 - \rho_s) + f_y \rho_s} \quad (10) $$

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seismic load if the rebar has the specific limit according to the applied condition [1]. Some scientist conducted the experimental study of the cyclic reinforced concrete structure to obtain the property and yield mechanism of the specimen when it receives the cyclic load.

Even if it has different mechanical properties than ASTM A706 with the requirements material retaining burden cyclic, some researchers experimented in testing the properties and a mechanical characteristic of SAS 670 as reinforcement material against the cyclic load. Hooman Tavallali is one of the researchers that conducted the study of properties and characteristic of R/C column with the reinforcement of SAS 670 that receive the cyclic load. The element of CC4-X beam and UC4-x are 2 of 7 specimens that resulted in stable and well enough hysteresis curve.

UC4-X is a specimen that uses SAS670 with grade 670MPa as the material of R/C column. And for CC4-X is the comparative specimen that is designed with grade 420MPa rebars. The hysteresis curve produced by both specimen of CC4-X and UC4-X shown in Fig. 11 has properties and characteristic that is slightly different. From the comparison between the hysteresis curve of the specimen CC4-X and UC4-X, it is seen that the reduction of the ratio of longitudinal rebar in increasing the steel strength will cause the post-cracking stiffness of the rebar decrease.

Besides, the increase of steel strength also caused the deformation of the column structure when the rebar experience the yielding for the first time become bigger.

According to hysteresis response curve from H. Tavallali in Fig. 12 and Fig. 13, the approach model of the curve that is used to represent the mechanical properties and characteristic hysteresis curve of r/c frame element building with high strength rebars in this study is using the response model with the type from Takeda Hysteresis.

C. Seismic Analytical Design Model

Hierarchy of the building collapse process can be obtained after do some a few analysis methods of the seismic-resistant building plan. Analysis of seismic load method that is used in this study is modal response spectrum analysis and inelastic dynamic non-linear time history analysis. Analysis and modeling of r/c building structure are using ETABS 2016 v.16.0.2 program.

1) Load Combination: For ultimate load combination that is used in the modal response spectrum analysis is refer to SNI 1726:2012 sections 4.2.2. And for the combination of ultimate load use in inelastic dynamic non-linear time history analysis according to non-linear dynamic procedure ASCE/SEI 41-13 section 7.2.2

\[
P = 1.4D \\
1.2D + 1.6L + 0.5L_r \\
1.2D + 1.6L_r + 0.5L \\
(1.2 + 0.2S_{DY})D \pm 1.0Q_{Ey} \pm 0.3Q_{Ex} + 0.5L \\
(1.2 + 0.2S_{DY})D \pm 1.0Q_{Ey} \pm 0.3Q_{Ex} + 0.5L \\
(0.9 – 0.2S_{DY})D \pm 1.0Q_{Ey} \pm 0.3Q_{Ex} \\
(0.9 – 0.2S_{DY})D \pm 1.0Q_{Ey} \pm 0.3Q_{Ex} \\
Q_g \pm 1.0E_x \pm 0.3E_y \\
Q_g \pm 1.0E_y \pm 0.3E_x
\]

where,

\[
Q_g = Q_D + Q_L + Q_S \tag{11}
\]

Combination load a – g is used for modal response spectrum analysis, and the combination load h – i is used for inelastic dynamic non-linear time history analysis.

For combination seismic load in inelastic dynamic non-linear time history analysis, Guidelines for Performance-Based Seismic Design of Tall Buildings (TBI), mention that application of ground motion acceleration as seismic load must use a pair of actual ground motion acceleration that works orthogonally in both main axes of the building.

2) Response Spectrum Analysis: The procedure of response spectrum analysis is done to control the movement of the fundamental natural period and obtaining the result of the elastic linear design of r/c elements structure in receiving the response spectra acceleration of the planned design.

Response spectrum analysis has to include enough modes to obtain of combined mass participation factor at least 90% of the actual mass for each orthogonal horizontal direction reviewed by the model. Effective seismic weight calculated in modal response spectrum analysis is the whole dead load that works in building structures. Live load of the floor is not calculated for public garage floor and open-air parking structure, so it considered not giving contribution for an effective seismic weight of the building structure.
Fig. 14 and Fig. 15 shows the dynamic movement of the structural model of building with rebar 550MPa and 690MPa mode shapes 3 degrees of freedom structure as follows:

Fig. 14  Three dynamic degrees of freedom R/C building model with steel reinforcement bar grade 550MPa

Fig. 15  Three dynamic degrees of freedom R/C building model with steel reinforcement bar grade 690MPa

The amount of base shear seismic of the structure acquired based on the acceleration response of the design of above the ground seismic spectrum that acts as a function of the fundamental natural period structure. The generally fundamental natural period for building structure uniform building determined by mode shape which has the lowest frequency or can also be called as fundamental mode shape. For the building structure with reinforced concrete rebar of 550MPa and 690MPa has a fundamental natural period as follows 4.089 seconds and 3.534 seconds.

TABLE IV

| Yield Strength $f_y$ (MPa) | Type of Beam | Section Name | Beam width $b_o$ (mm) | Beam Height $h_o$ (mm) |
|---------------------------|--------------|--------------|----------------------|----------------------|
| 550                       | Balok induk X | B48          | 400                  | 800                  |
|                           | Balok induk Y | B46          | 600                  | 800                  |
|                           | Balok Anak   | B36          | 800                  | 300                  |
| 690                       | Balok induk X | B79A         | 700                  | 850                  |
|                           | Balok induk Y | B57          | 550                  | 700                  |
|                           | Balok Anak   | B3A7         | 850                  | 350                  |

TABLE V

| Concentrated Comp. Strength $f_{c,y}$ (MPa) | Re-bars Yield Strength $f_y$ (MPa) | Type of Column | $R_0$ | $F_0$ | $D_b$ max (mm) | $h_c$ | $D_b$ | $c_x$ (mm) | $c_y$ (mm) | Section Name |
|-------------------------------------------|------------------------------------|----------------|-------|-------|----------------|-------|-------|-------------|-------------|--------------|
| 45                                        | 550                                | Segment A     | 0.617 | 32    | 23.723        | 900   | 900   | K91A        |             |              |
|                                           |                                    | Segment B     | 0.689 | 29    | 22.708        | 800   | 800   | KB8B        |             |              |
|                                           |                                    | Segment C     | 0.693 | 25    | 22.654        | 850   | 850   | KB8A6AC     |             |              |
|                                           | 690                                | Segment A     | 0.757 | 52    | 28.831        | 1000  | 1000  | K101A       |             |              |
|                                           |                                    | Segment B     | 0.592 | 35    | 26.226        | 800   | 800   | KB8B        |             |              |
|                                           |                                    | Segment C     | 0.653 | 22    | 25.017        | 700   | 700   | K17C        |             |              |

Fig. 16  Response spectrum design with an upper limit calculated period of grade 550MPa and 690MPa R/C building structure models

3) Earthquake Ground Motion: In an inelastic dynamic nonlinear time history analysis, the seismic loads that used are the actual ground acceleration from the location with similar geological, topographical, and seismic tectonic condition as the location of the actual building. The location of the building is in Denpasar, Bali, with intermediate ground classification. The SNI1726:2012 is giving limitation that at least three appropriate ground motion of acceleration must be used in inelastic dynamic nonlinear time history analysis. In this study, the inelastic dynamic nonlinear time history analysis is using a several recording of actual ground acceleration acquired from PEER Ground Motion Database (Pacific Earthquake Engineering Research Center).
The recommended value of the ground acceleration magnitude for inelastic dynamic nonlinear time history analysis ranged between 4.9 - 7.9 SR. The average shear wave velocity in the depth of 30m ($v_{s,30}$) of actual ground acceleration is similar to the classification of the ground structure of the building. For intermediate ground classification, SNI 1726:2012 giving limitation for average shear wave velocity in the depth of 30m ranged between 175 - 350 m/s. The distance range of the causative fault and the source of the earthquake from the actual ground acceleration should be adjusted with the condition of the causative fault and the source of the earthquake in Indonesian area which indeed dominated by the epicenter that came from causative fault on the sea area [25].

The ground motions which have been chosen to represent the ground motion of the building location reviewed must be scaled in a way so the average value of the spectrum response with 5% damping of all the ground motions cannot be less than the maximum acceleration seismic response spectrum MCE on the surface of the ground for period of 0.2T to 1.5T, in which T is natural structural period in variety of fundamental natural period to the direction of analyzed response.

The scale of ground motion acceleration that is used for the inelastic dynamic nonlinear time history analysis on reinforced concrete building in this study is a scale with two dimension analysis (2D), wherein each pair of horizontal ground motion component only one with the biggest pseudo-spectra intensity is used.

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![MCER Response Spectrum at Stiff Soil Site Class, Denpasar, Bali](image)

The scale of ground motion acceleration that is used for the inelastic dynamic nonlinear time history analysis on reinforced concrete building in this study is a scale with two dimension analysis (2D), wherein each pair of horizontal ground motion component only one with the biggest pseudo-spectra intensity is used.

![The Average of Pseudoacceleration R. Spectra of Grade 550MPa Structure Model](image)

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The scale of ground motion acceleration that is used for the inelastic dynamic nonlinear time history analysis on reinforced concrete building in this study is a scale with two dimension analysis (2D), wherein each pair of horizontal ground motion component only one with the biggest pseudo-spectra intensity is used.
III. RESULT AND DISCUSSION

A. Hinges Properties

Seismic performance building structure generally designed for a lower seismic load than spectra target. This is possible because the structure is designed to experience damage or inelastic behavior through the construction of plasticity joint of the structure elements when it received design seismic load.

In nonlinear dynamic procedure (NDP) arranged in either FEMA 356 or ASCE/SEI 41-13, the capacity of plastic hinges shown by the relationship curve between bearing capacity (flexure or lateral) against deformation (rotation or displacement) which resulted in the structural element. Plastic hinge model uses elastoplastic (bilinear) curve where the yielding point used is significant yield moment coordinate point. The general curve of the model of the plastic hinge generally described through five important points (A, B, C, D, and E) in accordance to the parameter arranged on the table 10-7 and 10-8 ASCE/SEI 41-13.

\[ \theta_p = (\varphi_u - \varphi_y) l_p \]  

where,  
\[ \varphi_u = \text{Ultimate Curvature} \]  
\[ \varphi_y = \text{Significant Yield Curvature} \]  

\[ l_p = 0.08 l + 0.022 d_b f_y \] (MPa)  

where \( l \) is the length of the clear span.

B. Component Performance

1) Acceptance Criteria: Acceptance criteria of component performance arranged on the table 10-7 and 10-8 ASCE/SEI 41-13 only applicable to the reinforced concrete element structure with the rebar strength less than 420MPa.

Acceptance criteria of component performance determined based on moment-rotation relationship curve obtained from the result of an independent analysis that follows acceptance criteria regulation that arranged in ASCE 41-13 section 7.6.3.
2) Assessment of Component Performance: The intensity of seismic load worked at a structure will be dissipated through an event of plastic hinge formation which occurred on the structural elements.

The hierarchy of structural collapse caused by the intensity of strong seismic load which is an equation of time determined based on the sequence of the plastic hinges formation which happened on the structural system. An ideal plastic mechanism of the building structure moment resisting frame system and which resulting in stable hysteretic behavior is a plastic mechanism of the beam's structural element (beam mechanism). In this mechanism, plastic hinge formed at the edge of beam structural element and on the base of the lowest column that meets with the foundation.

TABLE VII

| Actual Ground Motion | Direction of Load | a HTT coefficient | Convergence Total Step | Numbers of Step | Yield Mechanism |
|---------------------|-------------------|-------------------|-----------------------|----------------|----------------|
| "Chi-Chi_Taiwan"    | X                 | 0                 | 800                   | 800            | beam mechanism |
|                     | Y                 | 0                 | 800                   | 800            | beam mechanism |
| "Imperial Valley-06"| X                 | 0                 | 350                   | 350            | beam mechanism |
|                     | Y                 | 0                 | 150                   | 150            | beam mechanism |
| "Kobe_Japan"        | X                 | 0                 | 400                   | 400            | story mechanism |
|                     | Y                 | 0                 | 195                   | 195            | story mechanism |
| "Loma Prieta"       | X                 | 0                 | 900                   | 900            | beam mechanism |
|                     | Y                 | 0                 | 550                   | 550            | beam mechanism |
| "Northridge-01"     | X                 | 0                 | 190                   | 190            | story mechanism |
|                     | Y                 | 0                 | 190                   | 190            | story mechanism |
| "San Fernando"      | X                 | 0                 | 400                   | 400            | beam mechanism |
|                     | Y                 | 0                 | 400                   | 400            | beam mechanism |

The yielding mechanism of building structures with 550MPa strength rebar dominated by beam mechanism, while building structural collapse mechanism with 690MPa strength rebar dominated by story mechanism. The maximum rotation that produces by the beam's structural elements can be used as a reference to determine the performance of the structural element.

C. Global Performance

1) Acceptance Criteria: The performance level of the structural system will decrease according to an increment of the displacement value which occurred at every story of the reinforced concrete building.

TABLE XI

| Actual Ground Motion | Direction of Load | Type of Beam | Story | Moment Ultimate | Maximum Plastic Rotation | Performance Level |
|---------------------|-------------------|--------------|-------|-----------------|--------------------------|------------------|
| "Chi-Chi_Taiwan"    | X                 | Storey5      | 64187.8 | 0.011512 | A to 10 |
|                     | Y                 | Storey5      | 230811.31 | -0.002047 | A to 10 |
| "Imperial Valley-06"| X                 | Storey5      | 645261.12 | -0.005632 | A to 10 |
|                     | Y                 | Storey5      | 228951.2 | -0.005891 | A to 10 |
| "Kobe_Japan"        | X                 | Storey4      | 478793.37 | -0.003621 | A to 10 |
|                     | Y                 | Storey4      | 441725.1 | 0.007026 | A to 10 |
| "Landers"           | X                 | Storey5      | 481024.33 | -0.009686 | A to 10 |
|                     | Y                 | Storey5      | 228856.46 | -0.005770 | A to 10 |
| "Loma Prieta"       | X                 | Storey5      | 481700.77 | -0.005114 | A to 10 |
|                     | Y                 | Storey5      | 184032.5 | -0.009944 | A to 10 |
| "Northridge-01"     | X                 | Storey4      | 499993.48 | -0.005339 | 10 to 15 |
|                     | Y                 | Storey4      | 392649.08 | 0.087635 | 10 to 15 |
| "San Fernando"      | X                 | Storey4      | 462739.37 | -0.004514 | A to 10 |
|                     | Y                 | Storey4      | 220575.02 | -0.00915 | 10 to 15 |

TABLE XII

| Actual Ground Motion | Direction of Load | Type of Beam | Story | Maximum Plastics Hinges Rotation of R/C Beam Element |
|---------------------|-------------------|--------------|-------|---------------------------------|
| "Chi-Chi_Taiwan"    | X                 | Storey3      | 934704.5 | 0.001152 | A to 10 |
|                     | Y                 | Storey3      | 647177.6 | -0.011384 | A to 10 |
| "Imperial Valley-06"| X                 | Storey4      | 189292.78 | -0.014867 | 10 to 15 |
|                     | Y                 | Storey4      | 455102.69 | -0.053941 | A to 10 |
| "Kobe_Japan"        | X                 | Storey5      | 1020331.1 | -0.003842 | 10 to 15 |
|                     | Y                 | Storey5      | 840178.31 | 0.006777 | A to 10 |
| "Landers"           | X                 | Storey5      | 95176.38 | -0.006286 | A to 10 |
|                     | Y                 | Storey5      | 951411.35 | -0.005754 | A to 10 |
| "Loma Prieta"       | X                 | Storey5      | 149956.13 | -0.016308 | A to 10 |
|                     | Y                 | Storey5      | 899661.35 | 0.103487 | A to 10 |
| "Northridge-01"     | X                 | Storey5      | 906011.5 | -0.024067 | A to 10 |
|                     | Y                 | Storey5      | 450680.59 | -0.006319 | 10 to 15 |
| "San Fernando"      | X                 | Storey3      | 1979281.03 | -0.11191 | 10 to 15 |
|                     | Y                 | Storey3      | 461397.74 | 0.014415 | A to 10 |
The parameter that can be used to provide performance level measurement of a structural building system is calculated by a drift ratio as an effect of seismic load design. There are two kinds of drift ratio value to be concerned in the global measurement of the reinforced concrete building performance level which is: maximum total drift and roof drift ratio.

The average result of the drift ratio caused by the seven pairs of actual ground motion acceleration is used as a reference to determine the global performance of building system structure. Because of that, performance level that is generated through global performance analysis can represent the performance level system structure.

2) Interstory Drift Ratio: From the result of the average interstory drift ratio analysis that has been done, the level global performance for both building structure with 550MPa and 690MPa strength rebars is still in the limit of interstory drift ratio IO to LS (Damage Control).

![Fig. 24](image1.png) The average of interstory drift ratio from R/C building structure with grade 550MPa steel re-bar caused by the seismic load at X and Y direction

![Fig. 25](image2.png) The average of interstory drift ratio from R/C building structure with grade 690MPa steel re-bar caused by the seismic load at X and Y direction

The biggest interstory drift ratio which occurred in the building structure with 550MPa and 690MPa strength rebars are as follows: 0.01729 and 0.0182, it almost exceeds the interstory drift ratio life safety limit (0.02).

Both are produced as an effect of actual ground motion acceleration seismic load towards Y with deformation movement through negative Y global axis of the building structure.

3) Roof Drift Ratio: The deviation value that issues for the analysis is the biggest displacement value that occurred at the center of mass roof story level as a result of the seismic distribution base shear load on the story which produces through a variety of structural mode shape natural period response as long as the actual ground motion acceleration seismic load occurs.

From the average roof drift ratio analysis result that has been done, the level global performance for both reinforcement concrete building structure with 550MPa and 690MPa strength rebars are still within the limit of roof drift ratio IO to LS (Damage Control).

The biggest roof drift ratio that occurs in the building structure with grade 550MPa is 0.00843 caused by actual ground motion acceleration towards Y direction with deformation movement to the global axis of negative Y from the building structure. The biggest roof drift ratio that occurs in the building structure with grade 690MPa is 0.0084 caused by actual ground motion acceleration towards Y direction with deformation movement to the global axis of positive Y from the building structure.

IV. CONCLUSIONS

From the result of literature studies and performance analysis that is done related to seismic performances 550MPa and 690MPa high strength rebar in R/C building, we can conclude important things, which are: Preliminary the section dimension of R/C element structure produce bigger size than preliminary using normal rebar strength, both for R/C beam and column. The parameter of plastic hinges moment-rotation relationship curve and acceptance criteria of high-strength R/C element performance has to be determined through moment-curvature analysis of the section independently. The mechanism of the building structure collapse with grade 550MPa is dominated by beam mechanism. But for the mechanism of the building structure collapse with grade 690MPa is dominated by story...
mechanism. Global performance of high rise r/c building with grade 550MPa and 690MPa is in the level of performance damage control. The concentration of energy dissipation of biggest seismic load by the r/c beam happens in 4th and 5th floor, both in high rise r/c building with grade 550MPa and 690MPa. The material of high strength rebar can be used in special moment frame r/c building design if supported by the result of experiment data and accounted structure analysis.

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