Assessment of Korean Seismic Design Practice for Low-rise Reinforced Concrete Wall-frame Buildings

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Abstract
Korean seismic design practice for ordinary reinforced concrete wall-frame buildings is similar to but a little different from the equivalent lateral force design procedure per ASCE 7. The specific feature of the Korean practice is not to separate the walls and frames into an independent structural system, but to analyze and design them simultaneously, while they are separated and then analyzed and designed individually if following ASCE 7. Therefore, the Korean practice is assessed by using the performance evaluation procedure presented in FEMA P695. The results showed that the Korean design practice is not always applicable to all RC wall-frame buildings with ordinary shear walls. This negative result is due to a significant reduction in the design moment of the walls. The reduced moment makes the thickness and vertical reinforcement of walls designed by the Korean practice significantly smaller than those by ASCE 7, while the frame member sizes are similar. If structural engineers want to keep using the same design practice, appropriate methods to offset the significant reduction in the design moment should be incorporated such as reducing the R-factor or limiting reduction in the design moment.

Keywords: wall-frame building; RC shear wall; seismic design; low-rise RC building; seismic performance

1. Introduction
Many reinforced concrete (RC) buildings are composed of walls and frames in Korea. The walls are very effective in resisting seismic lateral force and have a role in architectural elements, such as stairways or elevator shafts, while frames resist most gravity loads. Structural engineers can therefore easily select walls as a major seismic-force-resisting system (SFRS) in buildings composed of both walls and frames. Therefore, the subject of this study is related to Korean design practice for the walls in buildings. It is noted that the object of this study is limited to RC ordinary systems because the typical Korean practice is limited to the design of such systems. In addition, while there exist various design procedures, this study is focused on the equivalent lateral force (ELF) design specified in codes such as Korean Building Code (KBC) (2016) or ASCE 7 (2010) because it is the most popular and simplest way of designing the ordinary system.

For the design of ordinary RC shear walls as part of wall-frame buildings, one can choose one of two design options when following the codes. One can be chosen from an ordinary shear wall within the building frames system (BFS), and another can be chosen from that within the wall-frame interactive system (WFIS). Table 1. presents key seismic design parameters (response modification factor $R$, system over-strength factor $\Omega_o$, and deflection amplification factor $C_d$), for both options specified in ASCE 7. The differences between the two approaches are basically in the level of the $R$ and $C_d$ factors, as well as the allowable range of the seismic design category (SDC). The design parameters in KBC are identical to those in ASCE 7 except for the relaxed allowable range. According to KBC, there is no limit for ordinary shear walls with a height of less than 60 m even at SDC D regardless of the system.

Table 1. Seismic Design Parameters for Ordinary Shear Walls in ASCE 7 (2010)

| System                        | $R$ | $\Omega_o$ | $C_d$ | Limit   |
|-------------------------------|-----|------------|-------|---------|
| Building Frame System         | 5   | 2.5        | 4.5   | SDC A, B, C |
| Wall-Frame Interactive System| 4.5 | 2.5        | 4     | SDC A, B  |

When structural engineers want to design ordinary shear walls in an RC wall-frame building, they should decide which option will be suitable based on their experience. If choosing BFS, then the shear walls are considered to resist 100% of the ELF design lateral load, and the frames to resist only the gravity loads. If
choosing WFIS, the shear strength of the shear walls should be at least 75% of the design story shear at each story, while the frames should be able to resist 25% of the design story shear in every story. That is to say, the walls in the wall-frame buildings should resist or be designed for a specific portion of the design lateral load. To design the walls and frames for the specific portion of the design lateral load, they should be separated, analyzed individually, and then designed for each portion of the design load specified in the codes. It is a normal procedure to apply the ELF design according to the codes.

KBC has simply been adopting the concepts of ASCE 7, so the SFRSs and the ELF procedure in KBC are identical to those in ASCE 7. Therefore, it is no wonder that Korean structural engineers also follow the normal procedure described in the preceding paragraph when they design according to KBC. However, they typically analyze a wall-frame building without separation of the walls and frames, and then design each member directly from the analysis results. This does not correspond to the normal procedure described in the preceding paragraph.

There are reasons why they do not divide a wall-frame building into walls and frames. One is the advancement of three-dimensional structural analysis programs. These analysis programs have made it relatively easy to analyze a building structure for all load combinations at once. Korean structural engineers utilize the analysis programs effectively. Another is the specific feature of a typical floor plan in Korea. It is difficult to separate walls and frames in wall-frame buildings.

Fig. 1 shows a typical floor plan for low-rise RC wall-frame buildings in Korea. It has only two or three bays in each direction, with the walls located at one of the corners, where the stairs and elevator are usually located. The few beams and columns do not have a regular layout where they are all connected to each other on a grid. As a result, the walls and frames are very difficult to separate structurally. If designing a building like that shown in Fig. 1, an ordinary shear wall within the BFS, the walls must be separated from the building, and analyzed independently for the design lateral load. However, it is not possible because the walls are a key part of the frames and substantially share in resistance to gravity load.

The issue is that Korean design practice uses seismic design parameters for the ordinary shear wall within the BFS per ASCE 7. As described above, the shear walls should be analyzed and designed independently for 100% of the ELF design lateral load per ASCE 7. If selecting the Korean practice, the interaction between the walls and frames is explicitly included in the determination of forces, which means the walls and frames resist the design lateral load in proportion to their rigidities. The simultaneous analysis in the Korean practice may significantly reduce the wall moment because the flexural rigidity of the frames can absorb some overturning moments, which may result in lighter wall designs in comparison to the BFS. It can be said that the Korean design practice is similar to that for a WFIS per ASCE 7 in terms of wall design. However, in the Korean practice, a different response modification factor (R-factor) from the WFIS, which is 5.0 for the BFS, is used and no check for 75% and 25% design story shears for walls and frames, respectively, is conducted.

Fig. 1. Typical Floor Plan of RC Wall-Frame Building in Korea

Based on this background, this study is aimed at assessing the Korean design practice for low-rise RC wall-frame buildings. To achieve this goal, example buildings with an ideal wall and frame arrangement have been selected and designed according to the Korean practice. Their performance is evaluated by following the procedure presented in FEMA P695 (2009). The feasibility of the Korean design practice for wall-frame buildings is provided based on the evaluation result.

2. Design of Example Buildings

The story number of the example buildings is selected to be five because elevators are usually placed in buildings with five stories and above, and their shafts usually consist of RC shear walls. The plan of the example buildings is shown in Fig. 2. Two walls are located at one of three bays in the longitudinal direction only. It is noted that only the longitudinal direction where walls and frames are mixed is considered.

The length of the bay is chosen to be of two types, which are 4 m and 6 m in the longitudinal direction while that in the transverse direction is fixed at 4 m. The bay length affects the design of walls by means of the total building weight, which is the main variable for design lateral load, and moment arm, which is a main variable for flexural strength, as well as the design of frames by means of moment and shear in beams and axial load in columns.

The example buildings are designed using two design options; one is to design them by exactly following the procedure per ASCE 7 and another is to design them by following the Korean design practice. The former is denoted as BFS and the latter as KDP. The design
procedures for both options have been described in the previous chapter. The example buildings are assumed to be located at the boundary between SDC C and SDC D because RC ordinary shear walls are permitted for SDC A, B, and C in ASCE 7. The MCE spectral response acceleration parameters for short periods ($S_{MS}$) and at 1 second ($S_{M1}$) are 0.75g and 0.30g, respectively, for the location.

Reinforcement ratios of vertical structural members are intended to be similar, which are around 0.4% for both vertical and horizontal reinforcement of walls, and 1.7% for longitudinal reinforcement of columns. The reinforcement ratios vary above and below those numbers. The size of these members is determined by trying to keep the reinforcement ratios. After the size and reinforcement of the vertical members are determined, those for beam members are determined where reinforcement ratio is between 0.6% and 1.2%. For RC member design, the nominal strengths of concrete and reinforcing steel are taken as 21 MPa and 400 MPa, respectively. The design dead load, including self-weight of the 150-mm-thick slab plus superimposed dead load, is 5.2 kN/m$^2$, and the design live load is 2.5 kN/m$^2$. The member sizes and reinforcement of the example buildings are presented in Table 2. and Table 3.

Reinforcement ratios for short periods are around 0.75g and 0.30g, respectively, for the location. The wall thickness of the KDP is smaller than that of the BFS. This is because frame action has more of an effect on absorbing the overturning moment than base shear. As shown in Fig.3.(a), there is little difference in design shear force. On the other hand, the design moment decreases significantly as shown in Fig.3.(b). This decrease could lead to a reduction in wall thickness. In the case of the KDP, the wall thickness and corresponding reinforcement are identical for both 4-m and 6-m bay lengths, as the minimum reinforcement (maximum reinforcement spacing) is enough to resist the design moment in both cases. In the case of a 4-m bay length, the difference between the two options is larger than for a 6-m bay length. This is due to whether the minimum reinforcement is applicable or not. For a 6-m bay length, on the other hand, the minimum reinforcement is applied in both options. This difference in the wall thickness affects the performance evaluation results, which will be presented later. The dimensions of beams and columns are nearly identical because they resist only gravity loads in both options.
currently approved in ASCE 7; thus, it requires the systems of interest to comply with all applicable design requirements in ASCE 7. The ability of the systems should achieve an intended seismic performance objective, which is the primary life safety performance objective by requiring an acceptably low probability of collapse of the system when subjected to maximum considered earthquake (MCE) ground motions defined in ASCE 7. The ability of the buildings designed by the Korean practice has been assessed by the following evaluation methodology presented in FEMA P695.

A key parameter for the performance evaluation procedure is collapse margin ratio (CMR). The CMR is the ratio of capacity to demand as shown in Eq. (1).

\[
CMR = \frac{\hat{S}_{CT}}{S_{MT}} = \frac{SD_{CT}}{SD_{MT}}
\]  

(1)

where \( \hat{S}_{CT} \) (or corresponding displacement, \( SD_{CT} \)) is the median 5%-damped spectral acceleration of the collapse level ground motions, and \( S_{MT} \) (or corresponding displacement, \( SD_{MT} \)) is the 5%-damped spectral acceleration of the MCE ground motions. The spectral acceleration of a collapse ground motion can be determined by incremental dynamic analysis (IDA) (Vamvatsikos et al., 2002) for each ground motion, and \( \hat{S}_{CT} \) is the median of the results for a number of ground motions. \( S_{MT} \) is the spectral acceleration at the fundamental period of the example building.

The CMR is adjusted by the spectral shape factor (SSF), which is a function of the fundamental period, period-based ductility, and applicable SDC. The adjusted collapse margin ratio (ACMR) is as shown in Eq. (2).

\[
ACMR = SSF \times CMR
\]  

(2)

To identify the achievement of the performance objective of a system of interest, acceptable values of the ACMR are needed. They are defined as \( ACMR_{10\%} \) and \( ACMR_{20\%} \), which are values of acceptable collapse probability, taken as 10% and 20%, respectively. They are based on total system collapse uncertainty, which is a parameter for reflecting the contribution of many sources of uncertainty to variability in collapse capacity and will be described later in detail. \( ACMR_{10\%} \) is the acceptable value for a group of buildings while \( ACMR_{20\%} \) is that for an individual building. This study did not cover a group of buildings, so \( ACMR_{10\%} \) was selected as the acceptable value of the ACMR.

4. Nonlinear Analysis of Example Buildings

4.1 Nonlinear Modeling of Example Buildings

Nonlinear analyses were conducted using the OpenSees structural analysis program (OpenSees, 2006) to investigate the performance of the example buildings. Two-dimensional (2D) analyses have only been conducted in the longitudinal direction of the floor plan, which is due to plan symmetry and for convenience in modeling. In 2D modeling in OpenSees, beam and column members are represented by line elements. The beam members are modeled by the "beamWithHinges" element, which uses a lumped hinge model to simulate nonlinear flexural behavior. The hinge can simulate pinching as well as stiffness and strength degradation in hysteretic behavior. The column members are modeled by the "nonlinearBeamColumn" element, where section properties are modeled by fiber section elements. For the fiber section, the areas, coordinates, and stress vs. strain relationships must be provided for the concrete and reinforcing bars. The fiber section models the actual dimensions and reinforcement of a member, so axial load and bending moment interaction is automatically reflected in the columns.

The wall members are modeled by the "MVLEM" element as shown in Fig. 4., which is a 2D multiple-vertical-line-element-model (MVLEM) for simulation of flexure-dominated RC wall behavior in OpenSees. The axial-flexural response of the MVLEM is simulated by a series of uniaxial elements or macro-fibers connected to the rigid beams at the top and bottom levels. In this study, macro-fibers are used like column members. The only difference between the MVLEM wall model and the column model lies in the stress-strain relationship for reinforcing bars: the relationship for the wall model is trilinear with strength drop while that for the column model is bilinear without strength drop.

Fig. 4. Two-Dimensional Multiple-Vertical-Line-Element-Model (OpenSees, 2006)
Shear response is represented by a shear spring located at height "ch" from the bottom of the wall element (Fig.4.(a)), which is assumed to be elastic. It is noted that shear and flexural responses of the MVLEMs are not coupled. Rotations (Φ) and resulting transverse displacements (Δ) are calculated based on the wall curvature (ϕ) corresponding to the bending moment at height "ch" of each element (Fig.4.(b)). A value of c = 0.4 is used, which was recommended by Vulcano et al. (1988). A more detailed explanation of this model is provided in the OpenSees website (OpenSees, 2006).

To test the appropriateness of the wall model, a cyclic response derived from an experiment involving an ordinary shear wall is selected, which is that of specimen WSH4 in Dazio et al. (2009). The specimen has no special boundary element at both ends of a wall section. As can be seen in Fig.5.(a), the specimen loses its strength when drift exceeds 1.5%, which is normally accepted for ordinary shear walls without a special boundary element. To simulate this strength drop, the strain-stress relationship for reinforcing bars is represented by a "Hysteretic" material model. The cyclic behavior from the MVLEMs, as shown in Fig.5.(b), is earned by adjusting the strain at the capping point.

4.2 Simulated and Non-simulated Collapse Modes

The collapse behavior of structural components should be simulated in analysis models to assess the performance of the example buildings. It is desirable that all significant deterioration modes contributing to the collapse behavior can be directly simulated in analysis models. The collapse determined by the component models is defined as the "simulated collapse (SC) mode" in FEMA P695. If it is not possible (or practical) to directly simulate the significant deterioration modes, FEMA P695 suggested an alternative method, which evaluates the collapse behavior indirectly by checking the structural response quantities measured in structural analyses. This is defined as the "non-simulated collapse (NSC) mode." The NSC mode usually provides conservative estimates of the collapse limits. This NSC mode approach is a practical method that can evaluate the effects of deterioration and collapse mechanisms when they are difficult to simulate directly in the analysis model. In this study, both the SC and NSC modes are utilized to assess the performance of the example buildings.

As described in the previous section, the collapse of the beams is directly simulated in the analysis model, so it is defined as the SC mode, which is the ultimate curvature of beam-ends. The collapse of columns is not directly simulated in the analysis model, so it is defined as story drift (NSC mode), which is described later in detail. Unlike columns and beams, the collapse of walls, which is a strength drop (SC mode), is directly simulated in the analysis model, but plastic rotation and story drift (NSC modes) are also incorporated in the definition of collapse. This is because the strength drop must be as smooth as possible owing to the
convergence issue of the analysis model.

The plastic rotations for walls are adopted from acceptance criteria presented in ASCE 41. As the collapse limit must be defined, it is natural to select acceptable plastic hinge rotations corresponding to the collapse prevention performance level. However, the plastic hinge rotations corresponding to life safety performance level in ASCE 41 are selected in this study for conservative assessment because ordinary RC shear walls are expected to have no residual strength due to relatively poor detailing.

Another NSC mode for walls is story drift. As described above, the specimen WSH4 in Dazio et al. (2009), which has no special boundary element at both ends of a wall section, loses its strength at a drift of 1.6%. It should be noted that axial load ratio for this specimen is 0.057. For failure of walls without special boundary elements, Wood (1991) showed that shear wall buildings, where the shear walls did not have special boundary elements, did not collapse during the 1985 Chile earthquake due to the large amount of walls. The large amount of walls keeps story drift demands to less than 1.0%, on the other hand, many shear wall buildings were severely damaged, and a few even collapsed, during the 2010 Chile earthquake. Massone et al. (2012) indicated that the reason was an increase in wall axial load ratio from 0.1 to 0.2, which is defined as the factored axial compression divided by the product of wall gross cross-sectional area and nominal concrete compressive strength. Based on results from the Chile earthquakes and other related experimental results (Dazio, 2009; Oesterle, 1976), and considering the axial load ratio of the walls in this study, the story drift limit as an NSC mode is determined to be 1.0% for ordinary RC shear walls without special boundary elements.

To determine the story drift limit based on column failure, the PEER structural performance database (PEER, 2003) is utilized. It is recognized from the database that the story drift limit is highly dependent on hoop spacing and axial load ratio. Even though feasible data are limited for this study, an attempt was made to find specimens with similar conditions to columns designed in this study. Three specimens have ultimately been selected, which are provided by Atalay and Penzien (1975), Nosho et al. (1996), and Lynn (1999). Specimens are No. 6S1 (Atalay et al., 1975), No. 1 (Nosho et al., 1996), and 2CMH18 (Lynn, 1999), where hoop spacings are 0.4d, 0.8d, and 1.0d (d = effective depth), and axial load ratios are 0.10, 0.34, and 0.28, respectively. Their ultimate drifts are 2.4% (No. 6S1), 1.7% (No. 1), and 0.5% (2CMH18), respectively. If assuming the column hoop spacing to be between 0.5d and 1.0d, the story drift limit based on column failure can be more than 1.0%. However, the story drift limit for walls has already been determined to be 1.0%, so 1.0% story drift is finally determined as the NSC mode for the example buildings.

5. Nonlinear Dynamic Analysis

5.1 Ground Motions

Nonlinear dynamic analysis is implemented to determine collapse intensity for a set of ground motions. For the nonlinear dynamic analysis, a set of 22 far-field ground motions is adopted from FEMA P695 in this study. Their event and recordings station, and site and source data are presented in Table A-4A and Table A-4B in FEMA P695. The 22 ground motion records are extracted from the PEER NGA Database (PEER, 2013), of which information and parameters are presented in Table A-4C. Each record contains two perpendicular horizontal ground motion data, so the total number is 44. Fig. 7 shows the response spectra of the twenty-two far-field record set. Ground motion records are scaled by using the procedure described in Appendix A of FEMA P695.

5.2 Incremental Dynamic Analysis

To determine the median 5%-damped spectral acceleration of the collapse level ground motions, $S_{c\gamma}$, the IDA is required. The IDA utilizes multiple dynamic analyses for a given ground motion record of increasing intensity until collapse occurs or the model otherwise reaches a collapse limit state. This process is repeated for a set of ground motion records of sufficient number to determine median collapse and record-to-record variability.

Fig. 8 shows an example of IDA curves of spectral acceleration versus maximum story drift. The methodology to determine the collapse in the original IDA is that the incremented slope of the IDA curve is less than 20% of the slope from an elastic analysis. This will be feasible only when the analysis model can capture every nonlinear behavior explicitly (all the collapse behaviors are defined by SC mode). As already described, it is not completely true, and as a result, the NSC modes described in Section 4.2 are also incorporated to determine the collapse.

6. Performance Evaluation

The performance of the example buildings is evaluated to determine whether the Korean design practice for wall-frame buildings is appropriate. The
evaluation has been conducted by checking only the $R$-factor used for the design of the buildings even though the procedure in FEMA P695 may also evaluate the overstrength factor and deflection amplification factor.

6.1 Evaluation of Total System Collapse Uncertainty

When determining collapse capacity, there inherently exist various sources of uncertainty contributing to its variability. The larger the variability, the larger the collapse margin ratio. Therefore, it is significant to consider all likely sources of uncertainty in determining the collapse. FEMA P695 presents four different sources of uncertainties: Record-to-record uncertainty (RTR), Design requirement uncertainty (DR), Test data uncertainty (TD), and Modeling uncertainty (MDL).

RTR is attributed to variability in the response of buildings to different ground motion records. FEMA P695 suggests a fixed value of RTR, $\beta_{\text{RTR}} = 0.40$, which is assumed for systems with the period-based ductility, $\mu_T \geq 3$. For systems with $\mu_T < 3$, values of RTR can be reduced based on the following equation:

$$\beta_{\text{RTR}} = 0.1 + 0.1 \mu_T \leq 0.40 \quad (3)$$

where $\beta_{\text{RTR}}$ must be greater than or equal to 0.20. The period-based ductility for the KDP is over 3.0, so $\beta_{\text{RTR}}$ of 0.40 is used, while that for the BFS is around 2.0, so $\beta_{\text{RTR}}$ of 0.30 is calculated using Eq. (3).

DR is related to the completeness and robustness of the design requirements. For the two design options, different levels of rating are assumed, which are "good" and "fair" for the BFS and KDP, respectively. The KDP is relatively low complete, robust, and confident as compared to the BFS. The resulting $\beta_{\text{DR}}$ values are 0.20 and 0.35 for the BFS and KDP, respectively.

TD is related to the completeness and robustness of the test data used to define the system. TD is closely associated with, but distinct from, MDL. MDL is related to the representativeness of the example buildings and capability of the analysis models. Both uncertainty sources are assumed to be rated "fair," so $\beta_{\text{TD}}$ and $\beta_{\text{MDL}}$ for TD and MDL, respectively, are all assigned to be 0.35 in this study.

As the four uncertainty sources described above are assumed to be statistically independent and lognormally distributed with median value of unity and lognormal standard deviation parameters, the lognormal standard deviation parameter, $\beta_{\text{TOT}}$, describing total collapse uncertainty, is given by

$$\beta_{\text{TOT}} = \sqrt{\beta_{\text{RTR}}^2 + \beta_{\text{DR}}^2 + \beta_{\text{TD}}^2 + \beta_{\text{MDL}}^2} \quad (4)$$

By substituting the variables in Eq. (4) for the values described above, $\beta_{\text{TOT}}$ values for the BFS and KDP are 0.612 and 0.726, respectively; these values are used to estimate the acceptable values of the ACMR.

6.2 Evaluation of $R$

Performance evaluation results for the example buildings are presented in Table 4. As described in Chapter 3, the appropriateness of the $R$-factor is determined by checking the ACMR and $ACMR_{10\%}$. The $ACMR_{10\%}$ is determined to be 2.20 and 2.53 for the BFS and KDP, respectively, based on Table 7-3 in FEMA P695. As they depend on the $\beta_{\text{TOT}}$, the value of acceptable collapse probability for the KDP is larger than that for the BFS. That is to say, the ACMR for the KDP needs to be larger than that for the BFS to achieve acceptable performance.

As presented in Table 4., KDP-4m only fails to satisfy the criterion because the ACMR is relatively low while $ACMR_{10\%}$ is relatively high. As can be seen in Table 2. and Table 3., the thickness and vertical reinforcement of walls in KDP-4m are significantly smaller than those in BFS-4m, while frame member sizes are similar, which is the main cause for the failure of KDP-4m. This result indicates that the Korean design practice, which analyzes and designs the walls and frames simultaneously but uses the seismic design parameters corresponding to the BFS in ASCE 7, is not always applicable to every wall-frame building with ordinary RC shear walls.

7. Conclusion

The Korean design practice for ordinary RC wall-frame buildings is assessed by using the performance evaluation procedure presented in FEMA P695. The

| Type     | $\delta_{\text{CM}}$ (g) | $\delta_{\text{CMR}}$ (g) | CMR  | $\mu_T$ | SSF | ACMR | $ACMR_{10\%}$ | Decision |
|----------|--------------------------|---------------------------|------|---------|-----|------|----------------|----------|
| BFS-4m   | 0.75                     | 1.80                      | 2.40 | 2.2     | 1.06| 2.54 | 2.20           | Pass     |
| KDP-4m   | 0.75                     | 1.60                      | 2.13 | 4.2     | 1.09| 2.33 | 2.53           | Fail     |
| BFS-6m   | 0.75                     | 1.73                      | 2.40 | 2.1     | 1.06| 2.44 | 2.20           | Pass     |
| KDP-6m   | 0.75                     | 2.00                      | 2.67 | 4.1     | 1.09| 2.91 | 2.53           | Pass     |
design procedure for wall-frame buildings in the Korean practice is similar but not identical to that in ASCE 7. The specific feature of the practice is not to separate the walls and frames into an independent structural system, but to analyze and design them simultaneously.

The results showed that the Korean design practice is not always applicable to all RC wall-frame buildings with ordinary shear walls. If structural engineers want to keep using the same design practice, appropriate methods to offset the significant reduction in the design moment should be incorporated, for instance, using reduced R-factor, placing limitations on reduction in design moment for walls, and increasing the wall thickness.

The negative result for the Korean design practice emerges from the study of a very limited number of example buildings. To establish an authentic design procedure for the Korean practice, broad archetype buildings that can cover requirements and conditions in design and construction practice shall be developed and analyzed. This will be the goal of an additional study that will follow the current one.

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