Reliability-Based Load Criteria for Structural Design:
Load Factors and Load Combinations

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ABSTRACT

The safety problem in structural engineering can be treated more rationally with probabilistic methods. These methods provide basic tools for evaluating structural safety quantitatively. Uncertainties in loads, material properties, and construction practice, which have been traditionally dealt with by empirical safety factors, can be taken into account explicitly and consistently in probabilistic safety assessment. Based on such methods, this paper introduces a study conducted to develop reliability-based load factors and load combination suitable for use with common reinforced concrete structures in Jordan. The selection of a probabilistic methodology for performing the necessary reliability analysis and the collection and examination of statistical data on structural resistance and loads were involved. The checking equation format for the proposed load criteria was selected, and the load factors and load combinations were computed using a constrained optimization procedure. Comparisons of reliabilities obtained using the proposed procedure with existing criteria were made.

1. INTRODUCTION

In recent years, concrete structural design has been moving towards a more rational and probability–based design procedure referred to as "limit states design". Such a design procedure takes into account more information than deterministic methods in the design of structural components. This information includes uncertainties in the strength of various structural elements; in loads and load combinations; and modeling errors in analysis procedures. Probability-based design formats are more flexible and rational than working stress formats because they provide consistent levels of safety over various types of structures. In probability–based limit-state design, probabilistic methods are used to guide the selection of strength (resistance) factors and load factors, which account for the variability in the individual resistance and loads and give the desired overall level of reliability. The load and resistance factors (Also called partial safety factors) are different for each type of load and resistance. Generally, the higher the uncertainty associated with a load, the higher the corresponding load factor; and the higher uncertainty associated with strength, the lower the corresponding strength factor.

Standards for structural design contain requirements to insure that structures perform satisfactorily under the effect of various loads. These provisions, which include load factors, resistance factors, allowable stresses, and deflection limits, have evolved more or less subjectively through extensive successful and unsuccessful professional experience, examination of available experimental data, theory, and judgment (Galambos et al., 1982). The load and load combination factors specified in the ACI 318-99 Code have remained the same since the 1950s. The American Society of Civil Engineers (ASCE), however, issued the ASCE 7 Standard on Minimum Design Loads for Buildings and Other Structures (ASCE, 1998). This standard specifies loads and load combinations with corresponding load factors based on a probabilistic analysis using the statistcal data on load and resistance parameters available (Nowak and Szerszen, 2003). The 2002 edition of the Building Code Requirements for Structural Concrete (ACI 318-02) specifies loads and load combinations consistent with ASCE 7.

In Jordan, such safety and serviceability criteria

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traditionally have been based on foreign codes (Assi, 2001). Consequently, these criteria do not insure consistent levels of safety and performance for different local structures. Moreover, such criteria may be inappropriate for designs that employ new materials or structural schemes where little basis for judgment may exist. The load factors and load combinations should have a probabilistic basis because of the consistent framework this approach provides for treating the uncertain (random) variables invariably present in structural design (Ellingwood et al., 1982). The general approach taken in this study was to:

- Estimate the levels of reliability implied by the use of various current design standards and specifications for common design situations in which performance generally is felt to be satisfactory.

- Select a format for the proposed criteria that balance theoretical consistency and appeal with ease of use in practice.

- Select a set of load factors and load combinations suitable for use with common reinforced concrete structures in Jordan.

In general, the basic approach to develop reliability-based design rules is first to determine the relative reliability of designs based on current practice. This relative reliability can be expressed in terms of either a probability of failure or a reliability index. The reliability index for structural components normally varies between 2 and 6 (Mansour et al., 1984). By performing such reliability analysis for many structures, representative values of target reliability (or safety) index can be selected reflecting the average reliability implicit in current designs. Based on these values and by using reliability analysis again, it is possible to select partial safety factors for the loads and the strength random variables that can be used as a basis for developing the design requirements.

Therefore, the main objective of this study is to calculate a new set of load factors and load combinations that is based on a probabilistic analysis using the currently available local statistical data in Jordan. These optimal factors will be employed in the development of design code of reinforced concrete structures in Jordan, so that the reliability of designed elements is consistent with a predetermined target level.

2. RELIABILITY–BASED DESIGN CRITERIA

The reliability-based design of concrete structures requires the consideration of the following three components: (a) loads, (b) structural strength, and (c) methods of reliability analysis. There are two primary approaches for reliability based design: (a) direct reliability-based design and (b) Load and Resistance Factor Design (LRFD). The direct reliability-based design approach can include both Level 2 and/or Level 3 reliability methods. Level 2 reliability methods are based on the first two moments of the random variable probability distribution (mean and variance) and sometimes with a linear approximation of nonlinear limit states, whereas Level 3 reliability methods use the complete probabilistic characteristics of the random variables. In some cases, Level 3 reliability analysis is not possible because of the lack of complete information on the full probabilistic characteristics of the random variables. Also, computational difficulty in Level 3 methods sometimes discourages their uses. The LRFD approach is called a Level 1 reliability method. Level 1 reliability methods utilize partial safety factors that are reliability based; but the methods do not require explicit use of the probabilistic description of the variables (Ayyub and Assakkaf, 2000; Ellingwood, 1979).

The conceptual framework for probability-based design throughout this study is provided by classical reliability theory. The loads and resistances are assumed to be statistical variables and the necessary statistical information is assumed to be available. While this approach furnishes a sound theoretical basis for the evaluation of structural reliability, a number of conceptual and operational difficulties in its use have led to the development of First-Order, Second-Moment (FOSM) methods, so called because of the way they characterize the uncertainty in the variables and the linearization underlying the reliability analysis (Galambos et al., 1982).

A mathematical model is first defined which relates the resistance and load effect variables for the specific limit state of interest:

$$g\ (X_1, X_2, ... X_m) = 0 \tag{1}$$

in which X_i = resistance and load variables, and failure is said to occur where $g < 0$ for the specific ultimate or serviceability limit state. Then the reliability associated with any given design is checked by identifying a point

$(X_1, X_2, ... X_m)$ satisfying Eq. 1 such that

$$X_i = \overline{X}_i - \alpha_i \beta \; \sigma_i \tag{2a}$$

$$\alpha_i = \frac{\sigma_i \dfrac{\partial g}{\partial x_i}}{\sqrt{\sum \left(\sigma_i \dfrac{\partial g}{\partial x_i} \right)^2}} \tag{2b}$$

The mean and standard deviation of X_i are $\overline{X}_i$ and σ_i, respectively, and β is defined as the reliability index. The partial derivatives in Eq. 2b are evaluated at the point defined by Eq. 2a. This point is called the checking or design point. It is found by searching iteratively for the direction cosines, α_i, which minimize β. The first-order-reliability method is very well suited to perform such a reliability assessment, the complete computational steps were described in (Ayyub, 1997). Data on both structural resistance and load variables are required in order to develop probability-based design criteria. The basic information required is the probability distribution of each load and resistance variable and estimates of its mean and standard deviation or Coefficient Of Variation (COV). The mean and coefficient of variation of these basic variables should be representative of values that would be expected in actual structures *in situ*.

The uncertainties used in the reliability analysis would include inherent statistical variability in the basic strength and load parameters, and additional sources of uncertainty that arise due to the modeling and prediction errors and incomplete information. Included in these modeling uncertainties would be errors in estimating the parameters of the distribution functions, mathematical idealizations of structural capacity and of the actual load processes, uncertainties in calculation, and deviations in the applications of the various load or material specifications (Ellingwood et al., 1982).

Means, COVs, and probability distributions for structural resistance have been determined from test data on the strength of materials and on dimensions of structural members (Assi, 2001). A representative sampling of these data which summarize results of research program conducted over different sites in Jordan is given in Tables (1 and 2). The reliability analysis is performed for reinforced concrete components such as beams and columns. Ordinary concrete is considered, and wide range of reinforcing bar diameters is included. The probabilistic characteristics and nominal values for the

strength and load components were determined based on statistical analysis and recommended values from other specifications (Tables 1 and 2). The deterministic expressions for resistance used in this study follow the ACI 318-99 Code. The parameters shown in Table (2) are bias factor (λ) and coefficient of Variation (V). For each design case considered, the mean value of resistance is calculated as a product of the nominal (design) value and bias factor. The standard deviation is calculated as the product of the mean and coefficient of variation.

3. RELIABILITY BASED LIMIT STATES DESIGN

The second approach of reliability-based design (LRFD) consists of the requirement that a factored (reduced) strength of a structural component is larger than a linear combination of factored (magnified) load effects as given by the following general format:

$$\phi R_n \geq \sum_{i=1}^{m} \gamma_i L_{ni} \tag{3}$$

where ϕ =strength factor, R_n =nominal (or design) strength, γ_i =load factor for the i*th* load component out of n components, and L_{ni} =nominal (or design) value for the i*th* load component out of m components. In this approach, load effects are increased, and strength is reduced, by multiplying the corresponding characteristic (resistance) and load factors, respectively, or partial safety factors. The characteristic value of some quantity is the value that is used in current design practice, and it is usually equal to a certain percentile of the probability distribution of that quantity. The load and strength factors are different for each type of load and strength. These factors are determined probabilistically so that they correspond to a prescribed level of reliability or safety. It is also common to consider two clauses of performance function that correspond to strength and serviceability requirements.

The difference between the Allowable Stress Design (ASD) and the (LRFD) format is that the latter uses different safety factors for each type of load and strength. This allows for taking into consideration uncertainties in load and strength, and to scale their characteristic values accordingly in the design equation.

The design criteria can be adjusted in a systematic way to achieve more consistent reliability for different design situations and to reflect the consequence of failure because of the explicit probabilistic treatment given to

uncertain variables in the design equation. The work described in this study is concerned with only ultimate limit states, which include bending moment capacity (for beams). Various structural elements and materials utilized in their construction that are within the scope of the ACI 318-99 code were considered in this study.

4. SELECTION OF FORMAT

Probability based limit states design customarily employs loads or load effects, which are multiplied by load factors, and resistances which are multiplied by resistance factors in a set of checking equations which have the general form

Factored resistance ≥ Effects of factored loads   (4)

A number of different safety or serviceability checking equation formats are possible (Ellingwood et al., 1980). The selection of the format should be guided by needs for simplicity and continuity with existing formats, as well as theoretical considerations. The load and resistance factor format most likely to be accepted by the design engineer in Jordan is that of Galambos and Ravindra (1978):

$$U = \gamma_D D_n + \gamma_Q Q_n + \Sigma \gamma_J Q_{nj} \qquad (5)$$

In which $\gamma_D D_n$=factored dead load*; $\gamma_Q Q_n$*=factored principal variable load; and terms $\gamma_J Q_{nj}$ =the factored arbitrary point-in-time loads. The individual time-varying loads under consideration must be rotated in the equation, each load taking the position of the maximum variable load, Q_n, while the remaining loads are assigned values, Q_{nj}. If computational simplicity is considered important, a few fundamental load computations may be specified explicitly for design as is done in current standards. In general, the load factors should be applied to the load prior to performing the analysis that transforms the load to a load effect. If the relation between load and load effect is linear, it makes no difference where the load factor is applied. However, it may be unconservative to apply the factor to the load effect in certain nonlinear problems.

The factored resistance in Eq. 3 can also be expressed in a number of ways. The method used in this study is the use of resistance factors applied to structural action. The factored resistance is defined as the product, ϕR_n, of nominal calculated resistance, R_n, and a resistance factor,

ϕ. The main advantages of this specification are that modeling errors in the calculation of nominal resistance and the mode and consequence of failure of the structural member can be reflected easily in the selection of ϕ. The disadvantage is that ϕ is not applied directly to the source of uncertainty (material strength, dimensions, analytical models, etc.). Consequently, if ϕ is specified as constant, it is more difficult to maintain constant reliability over all design situations. The situation is analogous to the use of one load factor rather than a factor for each uncertain load variable.

However, the following fundamental load combination were used in this study:

- $U = \gamma_D D + \gamma_L L$
- $U = \gamma_D D + \gamma_S S$
- $U = \gamma_D D + \gamma_L L + \gamma_{W_{apt}} W_{apt}$
- $U = \gamma_D D + \gamma_{L_{apt}} L_{apt} + \gamma_W W$

Where D =dead load; L = live load; S = snow load; W =wind load; γ = load factor to be applied to nominal load; U= effect of factored loads, and a.p.t = arbitrary-point-in-time value of appended variable.

5. RELIABILITY-BASED LOADING CRITERIA

Target reliabilities, referred to as β provide the basis for the calibration of the proposed probability-based design criteria against the safety level of current design practice. These values of β will serve as a basis for the determination of the load factors in the considered load combinations. However, target reliabilities for selecting load factors were established on a 50-year basis (Assi, 2001). For load combinations involving only gravity loads, β= 2.7; for those involving additive wind or snow loads, β=2.45. It should be emphasized that the target reliabilities are chosen solely for the purpose of enabling the load factors to be selected intelligently. This is to insure that with the set of load factors developed, it will be possible to develop resistance criteria to achieve designs that are similar, in an overall sense, to those obtained using current acceptable practice.

Several levels of rigor for reliability based design can be identified (CIRIA, 1977). For a given limit state and β level 2 methods employ safety checks at a number of discrete design situations. The implied load and resistance factors may be obtained from

$$\gamma_i = \frac{X_i}{X_{ni}} \qquad (6)$$

in which X_i=checking point value of load or

resistance (Eqs. 1 and 2) for a prescribed reliability β; and X_{ni}=nominal value of that load or resistance specified in the appropriate standard or specification. These γ_i would vary for different design situations (e.g., ratios of live load effects to dead load effect). Using the FOSM method together with the target β values and the statistical parameters assessed (Tables 1 and 2), it is possible to compute the resistance factors ϕ's and the load factors γ's, for various loading cases (Ayyub, 1997).

In Figures 1, 2, and 3, load and resistance factors correspond to the target reliabilities are chosen for the reinforced concrete beams in bending with grade 400 reinforcement (mainly used in building construction in Jordan), subjected to D+L, D+S and D+L+W load combinations, respectively. Note that in these Figures γ_L is associated with basic unreduced live load, L_o, and not with nominal live load, L. The influence area was set equal to 56m² to determine statistics of L_{apt}. As observed in these Figures, the load factors have relatively small sensitivity to the resistance statistics. It should also be noted that for the ranges of time-varying loads under consideration, the difference in the resistance factors is within 0.1. This means that the resistance factors are not very sensitive to the time-varying loads, especially when these loads are small. This fact seems to be the same for D+L, D+S or D+L+W combinations as shown in Figures 1, 2, and 3. In these combinations, the magnitude of γ_D appears to be independent of the magnitudes of time-varying loads. The value of γ_D, which is about 1.2, is low compared to the existing standards, due to the fact that the variability in D is small compared to the variabilities in other loads. Also, the live load factor γ_L in the D+L_{apt}+W combination is small. This is because L_{apt} used in this combination is much smaller than L used in the existing standards.

From these observations, the use of a single load factor γ_D for all load combinations can be suggested which will not cause any significant deviation from the target β. We can also uncouple the load and resistance factors because of the weak dependency between them.

Figures 1, 2, and 3 also show that the load factors of the time-varying loads increases as the corresponding loads increase, because the time-varying loads have higher variability than the dead load. Then, using a single factor for a time-varying load will cause a deviation from the ideal constant reliability. However, it is possible to select one set of load factors that minimizes the extent of this deviation when all likely combinations of that

particular load is considered (Ellingwood et al., 1980).

The process of selecting constant load factors for a given load combination is not unique (Galambos et al., 1982). Following the approach used by Ellingwood et al. (1980), which is based on two steps: (1) defining some function, $I(\phi, \gamma_i)$ measuring the "closeness" between the target reliability and the reliability associated with the proposed load and resistance factor set; and (2) selecting γ_i and ϕ so as to minimize this function. The function measuring the "closeness" is defined as follows:

$$I(\phi, \gamma_i) = \sum_i \left(R_{ni}^{II} - R_{ni}^{I} \right)^2 P_i \qquad (7)$$

Where, R_n^{II}=the required nominal resistance associated with target β and a given set of nominal loads; R_n^{I}=the nominal resistance resulting from the design equation, which prescribes a set of load factors that are constant for all load ratios; and P_i = the relative weight assigned to the *ith* load combination.

In this minimization process, the limit state function will be described by Eq. (5). The weights P_i assigned for the combinations have estimated (Ellingwood et al., 1980).

- **Gravity Loads**

The gravity load combinations, D+L and D+S, were considered first. The best estimate of the dead load factor, γ_D, is about 1.2. Setting γ_D=1.2 will be the first constraint in the optimization process. Using reliability analysis, R_n^{II} was determined for the selected target reliabilities (β=2.7) and the optimum sets of (ϕ,γ_L) and (ϕ,γ_S) were determined by minimizing Eq. 7 for the two grades of reinforcement. Table (4) lists these factors. The final step is to select one load factor γ for both live and snow loads and calculates the corresponding ϕ factors. This will be another constraint to the optimization process. Here, γ_L is associated with L_o. The most appropriate factor here seems to range between 1.2 and 1.4, whichever is the closest to the optimum. However, if the 1.4 value is chosen, we shall get a low resistance factor in the range of 0.75 to 0.8, which may not be acceptable by the designers. To increase the resistance factor to the range of 0.8 to 0.9, the live (or snow) load factor should be increased to about 1.7. Thus, an optimum set of load factors for D+L or D+S combinations are as follows:

$U=1.2D+1.4L_o$ (8a)

$U=1.2D+1.4S$ (8b)

Whereas the load factors suggested for use in practice are:

$$U=1.2D+1.7L_o \tag{9a}$$
$$U=1.2D+1.7S \tag{9b}$$

It should be noted that the ϕ factor corresponding to Eqs. 9(a,b) are 0.8 and 0.87, respectively.

- **Wind**

Two alternatives of load combinations to be considered in this study (Galambos et al., 1982):

$$U= \gamma_D\, D + \gamma_{L_{apt}}\, L_o + \gamma_W\, W \tag{10a}$$

$$U= \gamma_D\, D + \gamma_L\, L_o + \gamma_{W_{apt}}\, W \tag{10b}$$

The maximum of these combinations will form the basis for design. Setting $\gamma_D=1.2$ the optimal load and resistance factors (ϕ, γ_i) were determined by first calculating R_n^{II} for both combinations based on the chosen target β using Eq. 7. These optimum values are listed in Table (5). As observed from these results, the rounded optimum factors are: $\gamma_{L_{apt}}=0.3$, $\gamma_W=1.4$ and $\gamma_L=1.4$, $\gamma_{W_{apt}}=0.15$. Corresponding to these rounded loads factors, ϕ equals 0.76 and 0.74 for Eqs 10(a) and 10(b), respectively. To increase ϕ above 0.8, it is appropriate to increase γ_w in Eq. (10a) to 1.5 and to increase γ_L in Eq. (10b) to 1.7. The 0.15 value for $\gamma_{W_{apt}}$ is very small and can be neglected. Then Eq. (10b) will be reduced to Eq. (9a), which correspond to the D+L. These results are shown in Table (5). Then the optimum sets are as follows:

$$U=1.2D+0.3L_o+1.4W \tag{11a}$$
$$U=1.2D+1.4L_o +0.15W \tag{11b}$$

The suggested sets to be used in practice are as follows:

$$U=1.2D+0.3L_o+1.5W \tag{12a}$$
$$U=1.2D+1.7L_o \tag{12b}$$

The ϕ factors corresponding to Eqs. 12(a) and 12(b) are 0.81 and 0.82, respectively.

6. SAFETY LEVEL OF RECOMMENDED DESIGN

The safety levels associated with the recommended load and resistance factors are shown in Figures 4 and 5 as a function of L_o/D and W/D for the D+L and D+L_{apt}+W load combinations, respectively.

In Figure 4 the ACI Code, optimum and proposed sets of load factors for the D+L combination are compared. A parallel trend is observed in this Figure of both the ACI Code and the proposed design criterion to have almost the same stability around the target reliability. On the other side, the design criterion corresponding to the optimum load and resistance factors appear to comply with the target reliability level.

In Figure 5 the ACI and the proposed designs are compared for the D+L_{apt}+W load combination. In this Figure, the proposed design appears to be more stable about the target reliability than the ACI design practice. The effect of the live load on this combination is small because of the rather small load factor of 0.3. Also in this Figure, for (W/D≤1.0), β increases as W/D increases and for W/D >1.6 it begins to decrease. It was concluded that in the initial part of the curve the dead load is dominant, which has less variability and a high mean to nominal ratio, while for high W/D ratio, the wind load that has a higher variability will be the major component.

7. CONCLUSIONS

This paper has summarized the development of a set of load factors and load combinations for use in the structural design of reinforced concrete in Jordan. Data obtained from a comprehensive analysis of statistical and probabilistic information on various types of structural loads and capacities have been used to calculate reliability indices associated with the existing design practice. The level of safety in terms of β implied by the current design practice in Jordan based on ACI code provisions for beams in flexure were evaluated. Optimal load factors corresponding to the selected target reliabilities were found to yield low resistance factors. To bring the resistance factors to fall close to the desired range (0.8-0.9) which would result in additional conservatism over existing practice, the following load factors are recommended for use in practice.

$$U=1.2\, D+ 1.7\, L_o$$
$$U= 1.2\, D+ 1.7\, S$$
$$U=1.2\, D+0.3\, L_o+ 1.5\, W$$

However, the differences between the load factors presented in this study and previous studies are mainly due to the differences in the collecting and evaluating statistical and probabilistic information on various types of structural loads (dead, live, snow, wind) and structural capacities (resistances).

Therefore, the load and resistance factors presented here are valid for the probability distributions assumed in this study based on the statistically collected data in Jordan.

Table 1. Statistics of Basic Parameters (Assi, 2001).

Basic Variable	λ mean-to-nominal ratio	v% coefficient of variation	Probability Distribution
Concrete compressive strength			
f_c' =20 MPa	0.87	18	Normal
f_c' =25 MPa	0.84	21	Normal
f_c' =30 MPa	0.80	23	Normal
f_c' =35 MPa	0.82	22	Normal
Steel yield strength Grade 400 MPa Grade 460 MPa Grade 250 MPa	1.09 1.18 1.30	12.5	Log-normal
Geometry Beam depth d Beam width b	0.975 1.0	7.3 5.5	Normal Normal
Reinforcement Area A_s	0.985	4	Normal
stirrup spacing s	1.07	3.6	Normal

Table 2. Statistical Parameters for Load Combinations (Assi, 2001).

Load Component	Arbitrary-point-in time load		Probability Distribution	Maximum 50-year load		Probability Distribution
	λ mean-to-nominal ratio	v% coefficient of variation		λ mean-to-nominal ratio	v% coefficient of variation	
Dead load	1.14	18	Normal	1.14	18	Normal
Live load	0.2	70	Gamma	1.10	25	Type I
Snow	0.48	0.35	Type II	1.01	17	Log-normal
Wind	0.0	0.0	-	0.97	0.24	Type I

Table 3. Statistical Parameters of Resistance.

Limit State	Type of Member	Details	λ mean-to-nominal ratio	V% coefficient of variation
Flexure (Assi, 2001)	Beams Grade 400 MPa Grade 460 MPa	f_c'=20,25,30,35 MPa (f_c' = concrete compressive strength)	1.01 1.081	17 17
Shear (Assi, 2001)	Beams Stirrups 250 MPa f_c'=20, 25 MPa	Moderate ρ_v Minimum ρ_v (ρ_v =shear reinforcement percentage)	1.15 1.12	23 26
Axial Load (Ellingwood et al., 1980)	Short columns, Compression failures Short columns, Tension failures	f_c'=21 MPa f_c'=35 MPa f_c'=21 and 35 MPa	1.05 0.95 1.05	16 12 12

Table 4. Optimal Load and Resistance Factors for Gravity Loads.

Beam characteristics	Load combination	Target β	Optimal values		Optimal ϕ γ_D=1.2 γ_L=1.4 γ_S=1.4	Optimal ϕ γ_D=1.2 γ_L=1.7 γ_S=1.7
			ϕ	γ_L , γ_S		
Grade 400	D+L	2.7	0.74	1.36	0.76	0.83
Grade 460	D+L	2.7	0.77	1.36	0.79	0.85
Grade 400	D+S	2.45	0.73	1.21	0.75	0.81
Grade 460	D+S	2.45	0.79	1.21	0.81	0.87

Table 5. Optimal Load and Resistance Factors for Gravity and Environmental Loads.

Load combinations	Target β	Optimal values			Optimal ϕ for the fixed load factor			
		ϕ	γ_L	γ_w	γ_L=0.3 γ_w=1.4	γ_L=0.3 γ_w=1.5	γ_L=1.4 γ_w=0.15	γ_L=1.7 γ_w=0.15
D+L_{apt}+W	2.45	0.75	0.34	1.53	0.76	0.81	-	-
D+L+W_{apt}	2.7	0.70	1.26	0.07	-	-	0.74	0.82

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