한국도로포장공학회 논문집
第5券1號 2003年 3月
pp. 1 ~ 10

LRFD 기법을 활용한 연성포장 설계방안에 관한 연구

Application of Load and Resistance Factor Design Format to Designing Flexible Pavements

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Abstract

The objective of pavement design, just as with the design of other structures, is to obtain the most economical designs at specified levels of reliability. Methods that yield designs with different levels of reliability are undesirable, and over the course of time design approaches in the U.S. and Europe have converged toward the Load and Resistance Factor Design (LRFD) format in order to assure uniform reliability. At present the LRFD format has been implemented in concrete, steel, wood and bridge design specifications. In this paper, reliability theories are used to illustrate the development of an LRFD format for Mechanistic-Empirical (M-E) design of flexible pavements as an alternative of its reliability module. It is shown in this paper that ten candidate pavement sections designed with a reliability level using the AASHTO design guide (1986) do not have uniform structural reliability in terms of pavement mechanistic distress such as fatigue cracking and the uniform reliability can be achieved by using the LRFD format.

Keywords : LRFD, reliability index, fatigue cracking, partial safety factors, flexible pavement design

요 지

도로포장 설계의 목적은 다른 토목구조물의 설계와 마찬가지로 정해진 신뢰성 수준에서 가장 경제적인 설계를 얻는데 있다 할 것이다. 미국과 유럽의 선진국가들의 설계법들은 전혀 상이한 신뢰성 수준을 갖는 설계결과들을 생산하는 설계법을 지양해가면서 세월을 두고 변천하여 균등한 신뢰성 수준을 보장할 수 있는 LRFD양식을 채용하는 방향으로 발전되었다. 현재, LRFD 양식은 콘크리트 구조, 목구조, 강구조 및 교량설계기준에 적용되어 있다. 본 논문에서 저자는 역학-경험적 포장설계법에 적용될 신뢰성 모듈의 한 대안으로 신뢰성이론을 사용하여 LRFD 양식을 개발하고자 하는 과정을 예시하였다. AASHTO 86설계법에 따라 동일한 신뢰성 수준을 갖도록 설계된 10개의 포장단면이 피로균열과 같은 역학적 포장손상 측면에서 볼 때 균등한 구조적 신뢰성을 보여주지 못하며 LRFD양식을 사용함으로서 그러한 균등한 신뢰성을 확보할 수 있다는 사실이 본 논문을 통해 예시되고 있다.

핵심용어 : LRFD, 신뢰지수, 피로균열, 부분 안전계수, 아스팔트 포장 설계

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1. Background

In the design of structures a margin of safety is introduced between the nominal value of the strength adopted in the calculations, and the nominal value of the load effect (e.g. stress, bending moment, shear, etc.). Over the past 30-40 years this concept has been incorporated in the structural design specifications in various ways (Beeby, 1994).

At present most structural design specifications have adopted the Load and Resistance Factor Design (LRFD) format in which safety is achieved by applying appropriate factors to the strength (R) and load effects (Qi) to ensure a margin of safety. These factors of safety are dependent on the degrees of uncertainties and influences of the relevant quantities, and on the desired level of safety. The magnitude of the load and resistance factors is established using probabilistic calculations. It is the intent of this paper to illustrate how an appropriate pavement design model using LRFD can be developed based on an important performance measure (limit state) such as fatigue life. The approach suggested here can be used to develop design models for other performance measures such as pavement rutting, joint faulting, joint spalling, etc.

The trend in pavement engineering today, more than ever before, is towards providing economical designs at specified levels of reliability. Current design procedures, which are at best empirical-mechanistic in approach, often fall short in new design situations where prior experience does not exist. In addition, there is an increasing awareness that the raw data, on which solutions are based, themselves exhibit significant variability. It is the aim of this paper to demonstrate how concepts of reliability theory can be applied to develop safety factors corresponding to target reliabilities. Reliability theory provides a rational framework for addressing uncertainties in evaluating the predicted performance of an engineering structure during its intended life. In the context of pavements, design reliability is defined as the probability that a pavement as designed will withstand the required/design number of load applications during the desired service life while maintaining both structural and functional integrity (AASHTO, 1993). None of the input variables in performance prediction can be determined with full certainty. The methods associated with reliability analysis formally address these uncertainties in the selection of a viable pavement design. The objective of reliability analysis is to provide a specific degree of confidence that the pavement will perform satisfactorily while being subjected to traffic and environmental loads during its service life. Chua, Der Kiureghian, and Monosmith (1993), and Easa, Shalaby, and Helim (1996) have proposed formal procedures for the reliability-based analysis of pavement performance. These reliability-based approaches are not readily suited for use in day-to-day design, but they can be used to develop design procedures that assure consistent reliability for different design situations. So, in summary a reliability-based design procedure provides the following advantages:

- More consistent reliability is attained for different design situations because the different variabilities of the various strengths and loads are considered explicitly and independently.
- The reliability levels can be chosen to reflect the

consequences of failures. For example, if it is felt that rutting is more critical than fatigue since it may affect driver safety, a higher reliability can be assigned to rut depth than to fatigue.

- It gives the pavement designer a better understanding of the fundamental structural requirements and of the behavior of the pavement structure in meeting those requirements.
- It simplifies the design process by encouraging the same design philosophy and procedure to be adopted for all materials of construction.
- It provides a tool for updating standards in a rational manner as more data becomes available.

2. Concepts of Reliability Anlysis

Since there are uncertainties in the major parameter values of pavements; such as layer moduli, layer thickness, traffic, etc., it is reasonable to define each parameter as a random variable either with its mean and standard deviation or its complete probability distribution specified. For illustrative purposes, fatigue life is considered as the performance measure of interest. The safety margin (SM) for fatigue life expressed in log scale is:

$$SM_{fatigue} = Log(FL_t)\text{-}Log(FL_T) \quad (1)$$

where:

FL_T = actual fatigue life; and

FL_T = required fatigue life.

The probability of failure with respect to fatigue life is as follows:

$$P_f = P\,[FL_t < FL_T] = P\,[SM_{fatigue} < 0] \quad (2)$$

Reliability is defined as 1-P_f, where P_f is the probability of failure. Theoretically, when the complete probability distribution of the underlying variable is known, the probability of failure can be determined by computing the probability density function (PDF) of $SM_{fatigue}$, f_{SM}, and calculating the area under the curve to the left of the origin as illustrated in Figure 1.

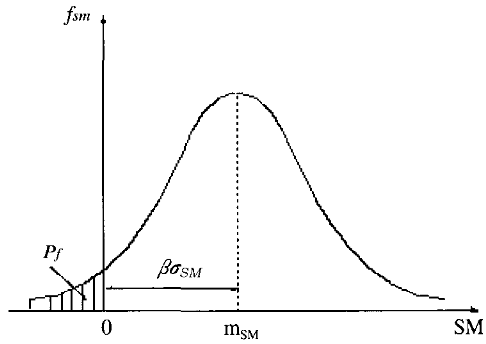

Figure 1. Probability Density Function of the Safety Margin

However, this is not possible if only partial probability information (e.g., the mean and standard deviation) of the design parameters is known. Even when the complete probability distribution of all parameters is known, computation of P_f is impractical if the performance measure of interest is non-linearly related to the basic variables governing the problem.

In practice, reliability is often measured by a reliability index β, which is defined such that:

$$P_f = \Phi\,(-\beta) \quad (3)$$

where Φ = cumulative standard normal distribution. As shown in Figure 1, if the SM is normally distributed, then β is the number of standard deviations of SM (σ_{SM}) separating the mean of *SM* (m_{SM}) from the failure point *SM*=0. The larger β is, the smaller the probability of failure (*e.g.*, β =0 corresponds to P_f=0.5 and β =3 corresponds to P_f =0.0013). For common civil engineering problems β is often computed by the mean value first order second moment (FOSM) method or the point estimate method (PEM) (Harr, 1987). Both methods produce consistent results for linear performance functions and symmetrically distributed random variables. However, for non-linear performance functions these methods can yield erroneous results that are inconsistent for mathematically different but mechanically equivalent failure criteria (*e.g* : W_t-w_T<0, W_t/w_T<1, or $log(W_t)$-$\log(w_T)$<0). To circumvent this problem, consistent and accurate estimates of the reliability index can be obtained by the first-order reliability method (FORM) proposed by Hasofer and Lind (1974) and Rackwitz and Fiessler (1978).

3. Reliability Associated with Aashto Designs

According to the AASHTO guide for design of pavement structures, reliability in pavement design is defined as the probability that a pavement section will perform satisfactorily for the design period. The conceptual equation for this definition is given by (AASHTO, 1993):

$$log\ W_t - log\ w_T = -Z_R S_o \qquad (4)$$

where:

W_t = allowable traffic (ESAL)

w_T = required traffic (ESAL)

Z_R = standard normal deviate for design reliability level; and

S_o = overall standard deviation to cover variations in section properties as well as traffic.

The characterization of reliability through Equation 4 assumes that W_t and w_T are log-normally distributed (i.e., *log Wt* and log w_T are normally distributed), and that all variations in the pavement cross-section and traffic are conglomerated into S_o. In standard practice, a constant value of S_o = 0.45 (as suggested by AASHTO) is used for flexible pavements. While it is theoretically possible to compute more representative values of S_o for specific design situations, this becomes a cumbersome task. The AASHTO procedure does not consider any particular mechanistic failure mode and therefore reliability of the pavement is addressed in general terms. In order to calculate W_t, the Design of New Pavement Structures software, DNPS86 requires the followings: pavement structure factors, such as base thickness; roadbed soil factors, such as roadbed soil resilient modulus; climate-related factors, such as drainage coefficients; and pavement condition factors, such as terminal present serviceability index.

Ten candidate (hypothetical) sections were designed using the DNPS86 software to select the AC thickness for assumed values of the AC resilient modulus (1380 MPa-3450 MPa) and subgrade resilient modulus (21 MPa-62 MPa), with the base thickness fixed at 230 mm and the base resilient modulus being fixed at 275 MPa. All candidate sections were designed to carry a 20 year traffic of 18.5 million ESALs with a reliability of 95% and an overall standard deviation S_o of 0.45. A summary of the DNPS86 designs can be found in Table 1.

Table 1. Pavement Cross-Section and Statistical Information

Test Section #	E_{AC} MPa.	E_{base} MPa.	E_{RB} MPa.	T_{AC} mm.
1	2760	275	62	254
2	2760	275	48	280
3	2760	275	34.5	305
4	2760	275	27.5	330
5	2760	275	21	342
6	1380	275	34.5	342
7	1725	275	34.5	330
8	2070	275	34.5	320
9	2760	275	34.5	305
10	3450	1275	34.5	266
COV	0.2	0.15	0.35	0.1

Note : 1. All the cross-sections were designed for a fatigue life (FL_T) of 18, 500,000 ESALs .
2. W_T is assumed to have a log-normal distribution with a COV of 0.42.
3. All variables are assumed to be normally distributed.

Table 2. Reliability Indices Computed Using Various Methods.

Section No.	FOSM	PEM	FORM
1	0.01	-0.02	0.03
2	0.77	0.62	0.66
3	1.35	1.20	1.23
4	1.78	1.74	1.75
5	1.95	1.97	2.00
6	1.26	1.19	1.17
7	1.22	1.13	1.16
8	1.24	1.11	1.12
9	1.35	1.20	1.23
10	0.63	0.67	0.69

The reliability indices for fatigue lives of the ten candidate sections designed by DNPS86, are computed using the FOSM, PEM and FORM methods. In this study the predicted fatigue life or resistance of a flexible pavement is expressed through (Huang, 1993) :

$$Log(FL) = 16.086 - 3.29 Log\left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854\left(\frac{E_{AC}}{10^3}\right) \quad (5)$$

where:

FL = load cycles to failure, ESALs

E_{AC} = resilient modulus of asphalt concrete (AC), and

ε_t = tensile strain at the bottom of the AC

The computed results are given in Table 2.

The distributions and Coefficients of Variations (COVs) shown in Table 1 for each variable were used in the analysis. The COVs used in the analysis represent as-built conditions (Huang, 1993). For any given section, there is no significant difference between the β-values computed by the three methods, which implies that the degree of non-linearity of the performance function is not pronounced. However, the difference in the β-values between the ten sections indicates that in general DNPS86 does not yield cross-sections with uniform reliability.

This motivates the development of a M-E design approach that tries to achieve uniform reliability for all designs and the LRFD format that is a particular form of the limit state design philosophy is one way to achieve this goal.

4. Development of Lrfd Criteria

The basic requirement in reliability-based design is that the reliability index β associated with an appropriate design equation should equal a target value β_0. The LRFD format is given by

$$\phi \ R_n \geq \Sigma \gamma_i Q_{n,i} \quad (6)$$

where:

ϕ = resistance factor,

γ_i = load factors,

R_n = nominal resistance, and

$Q_{n,i}$ = nominal load effects.

The ϕ and γ_i account for the variability of the resistance R and load effects Q_i, respectively. They also implicitly account for inaccuracies in the limit state models describing resistance and loads. In order to have a constant β for all design situations, the ϕ and γ_i must depend on the particular load combination and strength, and on the mean and standard deviation of all basic variables in the design equation. If a constant set of ϕ and γ_i are prescribed, the associated β will deviate from the target value β_o for certain design situations. However, it is possible to select one set of load and resistance factors that minimizes the extent of this deviation when considered over all likely combinations of load. While the resistance factors will depend on the material and limit state of interest, the load factors will be independent of these considerations and depend only on the variability of the loads. The overall design factors ϕ and γ_i are computed from partial safety factors associated with basic random variables that are required to characterize R and Q_i.

5. Illustrative Example of Lrfd

Allowable traffic volume in terms of the fatigue life may be expressed in generic form of equation (5) as

$$Log(FL_t) = f_R\,(T_{AC}, E_{AC}, E_{RB}, E_{base}) \quad (7)$$

since ε_t is a function of $T_{AC}, E_{AC}, E_{RB}, E_{base}$. The required fatigue life (or load effect) is taken to be

$$Log(FL_T) = log\,(18.5 \text{ million } ESALs\,) \quad (8)$$

The ten candidate sections designed by DNPS86 are used to develop an LRFD format for fatigue design. The tensile strain at the bottom of the AC was computed using the MICHPAVE computer program (Harichandran et.al., 1990). The other variables considered for the analysis were the thickness of the AC (T_{AC}), resilient modulus of the base and the AC (E_{base} and E_{AC}), and resilient modulus of the roadbed (E_{RB}). The information of these variables is given in Table 1.

The design criterion based on partial safety factors is given by:

$$f_R(\,\phi_T T_{AC},\ \phi_{AC}E_{AC},\ \phi_{RB}E_{RB},\ \phi_{base}E_{base}) \geq Log(\gamma_{FL_T} FL_T) \quad (9)$$

where:

$\phi_T,\ \phi_{AC},\ \phi_{base},\ \phi_{RB}$ = partial safety factor related to resistance; and

γ_{FL_T} = partial safety factor related to load (required traffic).

The partial safety factors ϕ_T and γ_{W_T} are calculated through (Ang and Tang, 1984) :

$$\phi_T\left(\frac{m_{T_{AC}}}{T_{AC,n}}\right)\left(1+\alpha_T\beta_O V_{T_{AC}}\right) \quad (10)$$

$$\gamma_{FL_T} = \frac{m_{FL_T}}{FL_{T,n}}\exp\begin{pmatrix}-0.5\ln(V_{FL_T}^2+1)+\\ \alpha_{FL_T}\beta_0(\ln(V_{FL_T}^2+1))^{0.5}\end{pmatrix} \cong \frac{m_{FL_T}}{FL_{T,n}}\exp(\alpha_{FL_T}\beta_0 V_{FL_T}) \quad (11)$$

where:

$m_{T_{AC}}$ = mean value of AC thickness,

m_{FL_T} = mean value of traffic,

$T_{AC,n}$ and $FL_{T,n}$ = nominal design values for T_{AC} and F_{LT},

α_T and α_{FL_T} = direction cosines for T_{AC} and FL_T associated with the so called design point in the FORM ; and

$V_{T_{AC}}$ and V_{FL_T} = coefficient of variation of T_{AC} and FL_T

The partial safety factors for ϕ_{AC}, ϕ_{base} and ϕ_{RB} are computed similar to ϕ_T. The expression for $(FL_T$ is different from that for ϕ_T because a log-normal distribution is assumed for FL_T, while a normal distribution is assumed for T_{AC}. Although Equation 5 is nonlinear, for many failure conditions encountered in civil engineering, such as Equation 9, the load and resistance factors obtained as described in the appendix are unique.

Partial safety factors, calculated using Equations 10 and 11 for the ten candidate sections are shown in Table 3. Equation 9 can be converted to the LRFD format;

$$\phi f_R(T_{AC,n}, E_{AC,n}, E_{RB,n}, E_{base,n}) \geq Log(\gamma_{FL_T} FL_T) \tag{12}$$

by computing a resistance factor ϕ associated with the overall resistance:

$$\phi = \frac{f_R(\phi_T, T_{AC}E_{AC}, \phi_{RB}E_{RB}, \phi_{base}E_{base})}{f_R(T_{AC,n}, E_{AC,n}, E_{RB,n}, E_{base,n})} \tag{13}$$

Suggested values for the overall resistance and load factors that yield approximately uniform reliability for all the design cases considered are given in Table 3.

Table 3 indicates that with increasing β_o, the partial safety factors associated with the resistance related variables decrease in magnitude and that associated with the load increases in magnitude. This is consistent since design values for resistance related variables (e.g. T_{AC}, E_{AC}, etc.) should be decreased while design load values (e.g. FL_T)

Table 3. Partial Safety Factors and Overall Resistance Factors for the 10 Candidate Test Sections at a Target Reliability Index (β) of 2.0 and 3.0

Test Section	β = 2						β = 3					
	ϕ_{AC}	ϕ_{base}	ϕ_{RB}	ϕ_T	γ_{W_T}	ϕ	ϕ_{AC}	ϕ_{base}	ϕ_{RB}	ϕ_T	γ_{W_T}	ϕ
1	0.88	0.92	0.99	0.85	1.60	0.940	0.82	0.88	0.98	0.78	2.02	0.910
2	0.89	0.91	0.98	0.86	1.65	0.940	0.83	0.87	0.97	0.79	2.12	0.910
3	0.86	0.91	0.99	0.87	1.63	0.940	0.79	0.87	0.98	0.80	1.97	0.910
4	0.87	0.92	0.99	0.85	1.53	0.930	0.80	0.88	0.99	0.77	1.89	0.900
5	0.89	0.92	0.99	0.85	1.54	0.930	0.83	0.88	0.99	0.77	1.91	0.900
6	0.91	0.89	0.99	0.86	1.61	0.940	0.87	0.83	0.98	0.79	2.40	0.909
7	0.91	0.90	0.99	0.85	1.56	0.941	0.86	0.85	0.98	0.78	1.96	0.906
8	0.89	0.90	0.98	0.87	1.65	0.941	0.83	0.85	0.98	0.80	2.11	0.912
9	0.86	0.91	0.99	0.87	1.63	0.941	0.79	0.87	0.98	0.80	2.08	0.911
10	0.87	0.93	0.99	0.85	1.57	0.941	0.80	0.89	0.98	0.78	1.97	0.908
Suggested Value					1.60	0.940					2.01	0.910

should be increased for increased safety levels.

The following two examples illustrate the application of the LRFD process to flexible pavement design. Sections 10 and 6 which had reliability indices of 0.69 and 1.17, respectively, when designed with DNPS86, were selected (the seed cross-sectional information is tabulated in Table 1 and the load and resistance factors can be found in Table 3). The objective of the LRFD design for these sections was to change the AC thickness and modulus so that the revised section would satisfy a fatigue life of 18.5 million ESALs at a target reliability of 2.0. Using a trial and error procedure, the AC thickness and modulus were revised until Equation 12 was just satisfied as illustrated in Figure 2. In order to assure that the reliability indices for the final sections were close to the target of 2.0, they were recomputed using the FORM. The properties of the two cross-sections selected by DNPS86 and LRFD are given in Table 4.

It can be seen that the LRFD does indeed yield cross-sections whose reliability indices are close to the target reliability of $\beta_o = 2$. The target reliabilities should be calibrated to inherent past practices associated with pavement design.

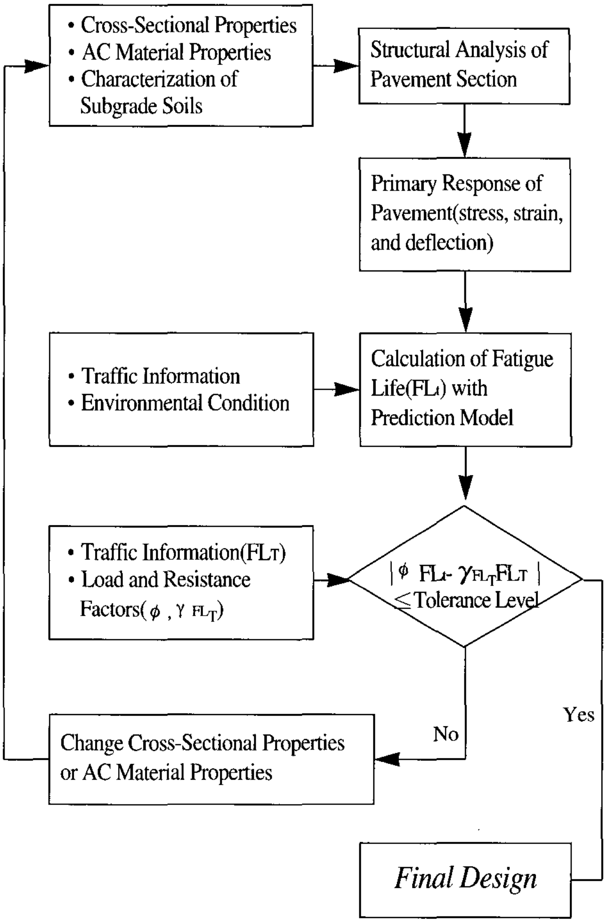

Figure 2. Flowchart for M-E Flexible Pavement Design Procedure Using LRFD Approach

Table 4. Properties of Sections Chosen by DNPS86 and LRFD

	Section 6		Section 10	
	DNPS86	LRFD	DNPS86	LRFD
T_{AC}, mm	342	370	266	330
E_{AC}, MPa	1380	2068	3450	3450
β	1.17	2.07	0.69	2.09

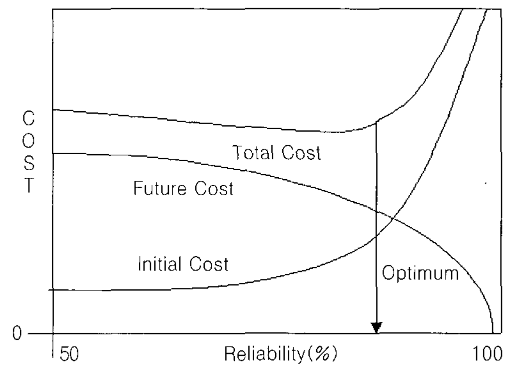

Figure 3. Approach to Identifying the Optimum Reliability Level for a Given Pavement (AASHTO, 1993)

In principle, an optimum target reliability index can be determined by performing a life cycle cost analysis (LCCA) as shown in Figure 3 (AASHTO, 1993). It is theoretically possible to reach the most economical target reliability by estimating initial cost and future cost including maintenance and rehabilitation costs, and establishing an optimal strategy. At the present time, however, it is reported that such an approach is impractical because the inference space over which a pavement design guide is being applied is much too large that the formulation of LCCA is difficult in practice (Brown, 1994). This means that for the time being, the most practical way to assign the optimal target reliability of the pavement is to depend on reasonable engineering judgment of experienced pavement designers (Kulhawy and Phoon, 1996).

6. Conclusions

A reliability-based LRFD design procedure that considers uncertainties associated with flexible pavement design is illustrated. It is demonstrated that the LRFD approach is capable of handling multiple design parameters and their associated uncertainties, and producing design outputs with an uniform reliability. By contrast, the AASHTO design procedure (as implemented in DNPS86) does not generally produce designs of uniform reliability for any particular mechanistic failure mode. Based on the proposed procedure, various load and resistance factors can be developed for different functional road classes such as interstate, principal arterials and residential streets for each performance measure. The proposed LRFD format, appropriately calibrated using much more data than used in this illustrative derivation, could be implemented in future design practice.

7. Acknowlegement

The authors are grateful to the Advanced Highway Research Center sponsored by Korea Science and Engineering Foundation and Hanyang University for its financial support.

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〈접수 : 2001. 8. 13〉