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Fourteenth International Specialty Conference on Cold-Formed Steel Structures
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A Study on the Flexural Behavior of Profiled Composite Beams

Gu Rok Yang[1], Young Seo Hwang[1] and Young Bong Kwon[2]

An analytical study on the behavior of composite beams, which are composed of cold-formed steel sheeting and normal strength concrete, is described. An analytical method to trace the nonlinear behavior of a composite beam is developed to include the nonlinear material properties of steel sheeting, reinforcing steel bar and concrete. However, since the method is complex and tedious to use, two simple formulas for the nonlinear moment-curvature relation of the composite beam have been proposed. A simple power model, which has been originally used to predict the flexural capacity of the beam to column connections, is proposed as the first formula. The second formula is composed of two experimental set of functions to express separately, the moment-curvature relation in the elastic and the plastic range. Both formulas have been proven to be accurate and useful for the design of profiled composite beams. The load-deflection behavior of the beams has been simulated by the step-by-step numerical integration method and is compared to available test results. The effects of the concrete cube strength and the thickness and strength of the cold-formed steel section on the flexural strength of the composite beam have also been studied.

INTRODUCTION

Cold-formed profiled steel sheeting has been widely used in the construction of composite slabs and used as a permanent formwork for non-composite slabs. The term "permanent formwork" indicates that the sheeting has not been removed after the hardening of the concrete material(Wright et al 1987). The advantages of using profiled sheeting for composite and non-composite slab construction included high degree of strength, ductility and material savings cost. Recent research has proven that the same advantages could be obtained in using profiled sheeting for composite beams(Uy and Bradford 1995, Oehlers et al 1994).

Uy and Bradford(1995) proposed a straightforward method to predict the moment-curvature relation of profiled composite beams, using the nonlinear stress-strain relations of both concrete and steel. However, the moment-curvature relation was not described in a simple formula which could be used to trace the nonlinear load-deformation behavior of composite beams. Concrete-filled steel box columns have been investigated experimentally and theoretically by Ge and Usami(1994). Simple formulas for the moment-thrust-curvature relation of the composite column have been proposed.

In this paper, the formulas proposed for the composite box columns by Ge and Usami are modified for the application to the profiled cold-formed composite beams. A simple power

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model, which was first applied to beam-to-column connections to predict the nonlinear moment-rotation behavior, is also proposed to simulate the moment-curvature relation of composite beams. A beam-deflection curve method, based on the step-by-step numerical integration procedures, has been developed to trace the nonlinear load-deflection curve of the composite beam. A parametric study is conducted to investigate the effects of the thickness and strength of the cold-formed steel section, and concrete cube strength. The numerical results, obtained by the proposed method, are compared with available test results.

MATERIAL PROPERTIES

The stress-strain relations of cold-formed steel sections generally shows gradual yielding since non-uniform plastic deformation of the cross section caused differences in the yield strength and residual stress of the fiber. The corner area generally shows more of a rounding curve than the flat area. The test yield strength of the section is generally higher than the nominal strength and the strength variation of the corner area is greater than that of the flat area. However, regardless of the position, the nonlinear stress-strain relations of profiled steel sheeting can be formulated by the Ramberg-Osgood formula as: (Eq. 1.1)

$$\varepsilon = \frac{\sigma}{E} + \varepsilon_p \left(\frac{\sigma}{\sigma_p} \right)^n \qquad \text{(Eq. 1.1)}$$

where E : elastic modulus of the linear portion

ε_p : plastic component of the strain (usually 0.2%)

σ_p : proof stress corresponding to 0.2% strain

n : the shape of the knee of the curve ($n = 25$: for cold-formed steels)

σ_p, ε_p, E, n obtained from Uy's tests and are given in Table 1 and Fig. 1.

Table 1. Material Properties of Steel Sheeting

σ_p[MPa(ksi)]	ε_p	E[MPa(ksi)]	n
552 (80)	0.002	205,000 (29,700)	25

The stress-strain relation of the deformed reinforcing steel bar can be assumed to be elastic-perfectly plastic. The stress-strain relation shows an inclined straight line with Young's Modulus up to the yield stress and then a horizontal line takes place, while neglecting the strain-hardening range of the properies. The yield stress and elastic modulus used by Uy and Bradford are given in Table 2.

Table 2. Material Properties of Reinforcing Steel

steel bar	σ_y[MPa(ksi)]	E[MPa(ksi)]
Y12	435 (63)	200,000 (29,000)

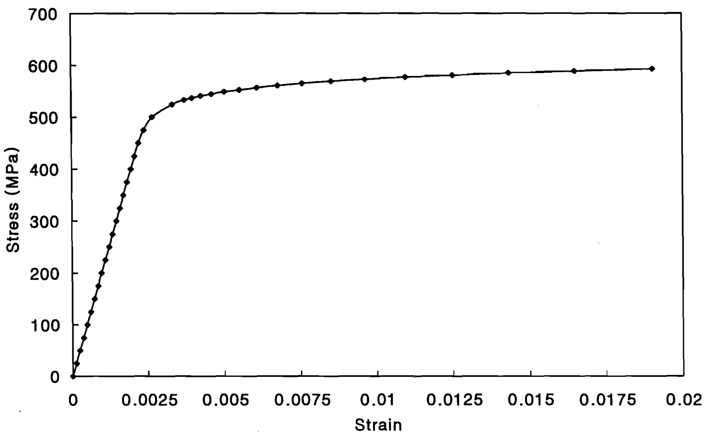


Fig. 1. Stress-Strain Relation of Profiled Steel Sheeting

The strength of concrete is generally expressed by the stress-strain relation curves of Hogenstad(1951) and Kent and Park(1971). These equations have been used for normal strength concrete. In this paper, the stress-strain relation proposed by Collins, et al(1993), which considered the brittle fracture of the high strength concrete, is used for the numerical model. The compressive stress and strain relation is given in (Eq. 1.2), Table 3 and Fig. 2.

$$\sigma = k_3 \cdot \sigma_{ck} \cdot \frac{\varepsilon}{\varepsilon_c} \cdot \frac{n}{n-1+\left(\frac{\varepsilon}{\varepsilon_c}\right)^{nk}} \qquad \text{(Eq. 1.2)}$$

where,

σ_{ck} : compressive strength of a concrete cube

$k = 1$ when $\frac{\varepsilon}{\varepsilon_c} < 1$ $\qquad$ $k = 0.67 + \frac{\sigma_{ck}}{62} > 1$ when $\frac{\varepsilon}{\varepsilon_c} > 1$

$k_3 = 0.6 + \frac{10}{\sigma_{ck}} \leq 0.85$ $\qquad$ $n = 0.8 + \frac{\sigma_{ck}}{17}$

$\varepsilon_c = \frac{\sigma_{ck}}{E_c} \cdot \frac{n}{n-1}$ $\qquad$ $E_c = 3320\sqrt{\sigma_{ck}} + 6900$ 1

Table 3. Material Properties of Concrete

σ_{ck}[MPa(ksi)]	ε_c	E_c[MPa(ksi)]
44.2 (6.4)	0.002161	28972 (4,205)
n	k	k_3
3.40	1.38	0.83

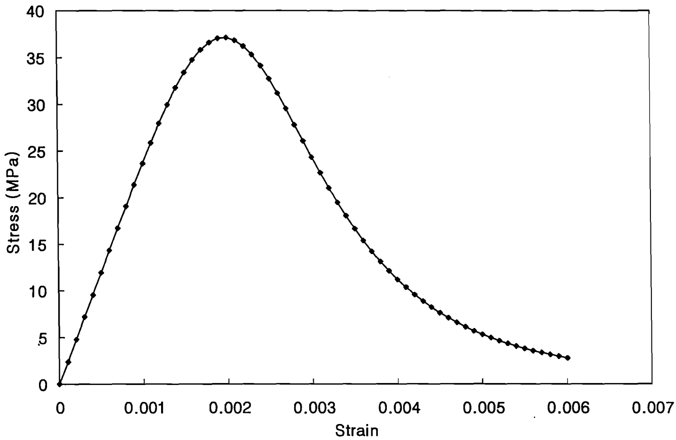


Fig. 2. Stress-Strain Relation of Concrete

The concrete tensile strength and strain relation proposed by Gilbert(1988), as a function of compressive strength, is chosen to include the tensile strength of the tension area of the composite beam section and expressed as: (Eq. 1.3a-c)

$$\sigma = \frac{\varepsilon \sigma_t}{\varepsilon_r}, \qquad 0 < \varepsilon \leq \varepsilon_r \ : \ \varepsilon_r = 0.1\% \qquad \text{(Eq. 1.3a)}$$

$$\sigma = \frac{(\varepsilon + \varepsilon_a)\sigma_t}{\varepsilon_r - \varepsilon_a}, \qquad \varepsilon_r < \varepsilon \leq \varepsilon_a \ : \ \varepsilon_a = 0.5\% \qquad \text{(Eq. 1.3b)}$$

$$\sigma = 0, \qquad \varepsilon > \varepsilon_a \qquad \text{(Eq. 1.3c)}$$

BUCKLING ANALYSIS OF STEEL SHEETING

The cold-formed steel section that has the same geometry and material properties as Uy's test section, was investigated, using an elastic buckling analysis based on the semi-analytical and the spline finite strip method(Hancock 1978, Lau and Hancock 1989). The numerical results of the section, which were assumed to be subjected to pure bending, are given in Fig. 3. The buckling modes of the steel section are shown in Fig. 4(a) and (b). As shown in Fig. 4(a) and (b), the buckled parts are restricted in the area above the top rib.

The semi-analytical analysis gives the buckling stresses at given half-wavelengths, whereas, the spline analysis gives the actual buckling stress of a given section length. As the beam length was increased, the top part of the simply supported beam sections, which were subjected to pure bending, were apt to buckle mainly in a local mode.

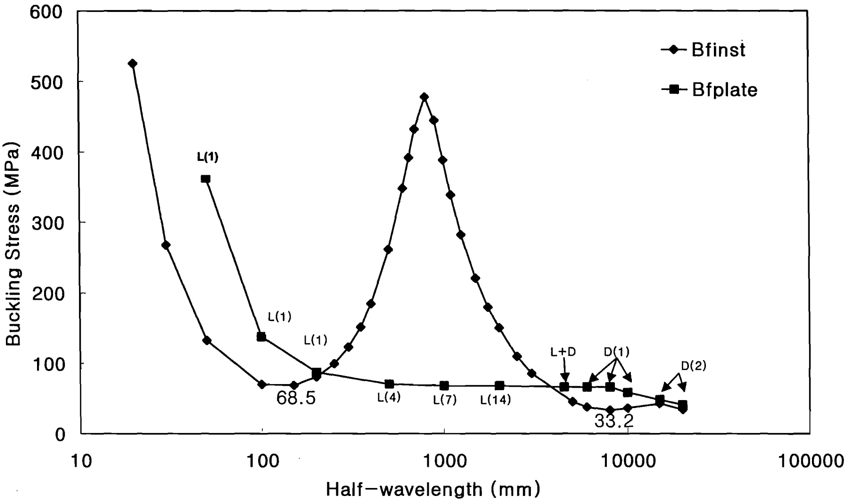
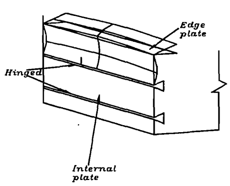
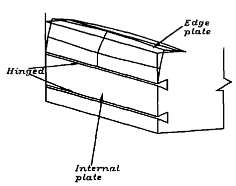


Fig. 3. Buckling Stress Versus Section Length / Half-Wavelength





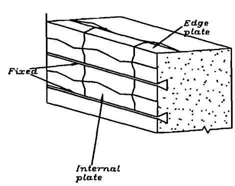


Fig. 4(a). Local Buckling of Steel Section

Fig. 4(b). Distortional Buckling of Steel Section

Fig. 4(c). Local Buckling of Composite Section

The boundary condition of the edge stiffener can be assumed to be clamped at three sides with one free edge. The top part above the top rib can be regarded as the plate with four clamped edges. The theoretical buckling coefficient of the plate, which is assumed to be clamped on all four edges or on three edges with one free edge, is 10.67 and 3.88 respectively (Timoshenko 1982). With these values, the buckling strength can be easily calculated using: (Eq. 2.1)

$$\sigma_l = \frac{K\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2 \quad \text{(Eq. 2.1)}$$

On the other hand, considering the local buckling mode of the steel section, the area above the top rib should be modeled as a plate assembly with linear varying compression and three clamped sides and one free edge. The buckling analysis results of the plate assembly are given in Fig. 5. The buckling strength of the steel section of the composite beam is much higher than that of the steel section. The section buckles mainly in a local buckling mode. The local buckling strain for an elastic local buckling mode is obtained by the local buckling strength divided by Young's Modulus of steel as: (Eq. 2.2)

$$\varepsilon_l = \frac{\sigma_l}{E_s} \quad \text{(Eq. 2.2)}$$

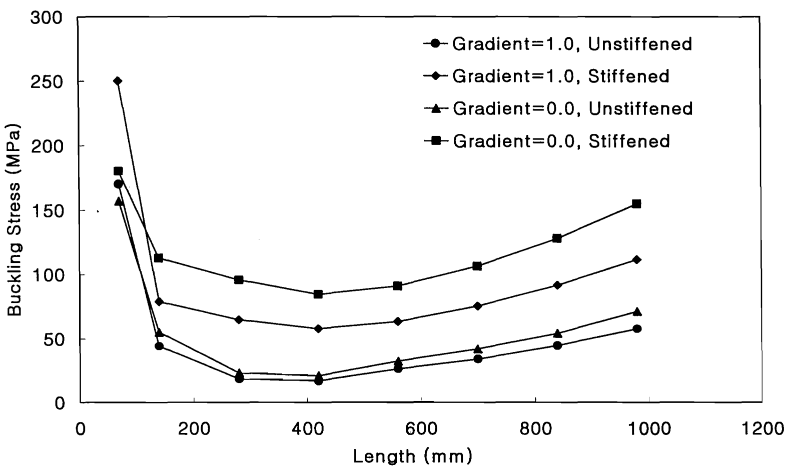


Fig. 5. Buckling Strength of the Plate with Three Edges Clamped, One Free Edge

MOMENT-CURVATURE RELATIONS OF COMPOSITE BEAMS

The section analysis method and simple formulas for the moment-curvature of the composite beam section are proposed and compared with the experimental results obtained by Uy and Bradford(1995). The section tested by Uy and Bradford is shown in Fig. 6. In the test, the one-third point loading is applied to ascertain the pure bending at the middle part of the composite beam. The moment-curvature relation is investigated via the strains obtained using Demek strain gauges.

Section Analysis Method

The method proposed is similar to that developed by Uy and Bradford(1995). The entire cross section of the composite beam is sub-divided into thin horizontal slices to allow the strain distribution to fit an idealized linear function. The effective width B_{eff} of the reinforced concrete beam is calculated using (Eq. 3.1). The cold-formed steel area of ribs is massed at the rib centroid location as shown in Fig. 6, for the simple calculation of the moment-curvature relation.

$$B_{eff} = B\left(1 - \frac{A_{voids}}{BD}\right) \quad \text{(Eq. 3.1)}$$

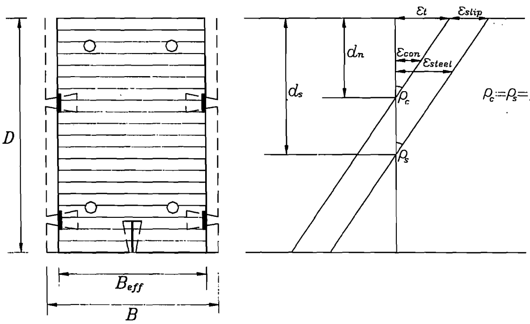


Fig. 6. Subdivided Cross Section

The calculation procedure to trace the moment-curvature relation of the composite beam section is as follows:

① Assume the strain ε_t at the top fiber.

② Neutral axis depth d_n is assumed and the strain distribution at each slice is calculated. The curvature is calculated with the top strain and neutral axis depth assumed, using: (Eq. 3.2)

$$\rho = \frac{\varepsilon_t}{d_n} \quad \text{(Eq. 3.2)}$$

③ The slip strain between the steel and concrete can be included as: (Eq. 3.3)

$$\varepsilon_s = \delta \rho d_s \quad \text{(Eq. 3.3)}$$

where δ is a slip strain parameter and d_s is the neutral axis of the steel section.

④ Calculate the stress at each slice with the stress-strain relation of cold-formed steel, reinforcing steel bar, and concrete.

⑤ Sum up the axial force of all slices.

⑥ Sum up the bending moment of all slices.

⑦ The bisection method is used to find the converged neutral axis depth for which the resultant axial force equals zero to a given accuracy.

⑧ Increase the top strain of the section and iterate procedures ②~⑦.

The results obtained by the section analysis method are given and compared with Uy's tests in Fig. 7. The comparison shows that the method produces quite reasonable results. It should noted that the slip strain should be decided appropriately to predict the accurate moment-curvature behavior of the composite beam. In this analysis, several values of the slip strain parameter have been tried and a slip strain parameter of 0.2 has been used to produce a reasonable curve fit.

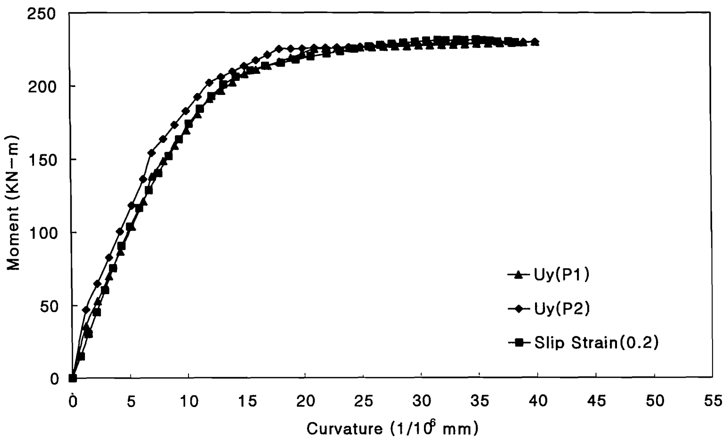


Fig. 7. Moment-Curvature Relation by Section Analysis

Modified Usami Formula

The moment-curvature relation which was originally proposed for the concrete-filled steel box column by Ge and Usami(1994) has been modified for the cold-formed steel composite beam. The formulas for the moment-curvature relation were expressed by two different equations, i.e. Eq. 3.4a and 3.4b for the elastic and plastic range of the materials respectively.

$$\text{<ELASTIC>} \quad \phi = \frac{m}{a_{00}} \qquad \text{(Eq. 3.4a)}$$

$$\text{<PLASTIC>} \quad \phi = -\frac{\ln \frac{m - m_{pc}}{c_{00}}}{b_{00}} \qquad \text{(Eq. 3.4b)}$$

where,

$$m = \frac{M}{M_{pc,0}}, \quad \phi = \frac{\Phi}{\Phi_y}, \quad p = \frac{P}{P_{yc}} \qquad \text{(Eq. 3.4c)}$$

in which,

$$M_{pc,0} = \frac{1}{2}(b-t)\,\eta_0^2\,\sigma_{ck} + [\,(b+t)\,d + \eta_0^2 + (d-t-\eta_0)^2\,]\,t\,\sigma_y$$

: plastic moment of the concrete-filled section when the axial force equals zero.

$$\eta_0=\frac{2(d-t)t\sigma_y}{(b-t)\sigma_{ck}+4t\sigma_y}$$

$\Phi_y=\dfrac{\sigma_y\, d}{2\, E_s}$: yielding curvature of the steel section

$P_{yc}=A_s\sigma_y+A_c\sigma_{ck}$: plastic axial force of the composite section.

The normalized values m_y, ϕ_y, m_{pc} and the parameters a_{00}, b_{00}, c_{00} are defined as:

$$a_{00}=\frac{m_y}{\phi_y} \quad \text{(Eq. 3.5a)}$$

$$b_{00}=\frac{m_y}{\phi_y\,(m_{pc}-m_y)} \quad \text{(Eq. 3.5b)}$$

$$c_{00}=-\frac{m_{pc}-m_y}{\exp\dfrac{-m_y}{m_{pc}-m_y}} \quad \text{(Eq. 3.5c)}$$

$$m_y=(1.0347R+0.2905)p+0.13339R+0.28615 \quad \text{(Eq. 3.5d)}$$

$$\phi_y=0.7070p+0.5329 \quad \text{(Eq. 3.5e)}$$

$$m_{pc}=1.0-(1+c)p^2+cp \quad \text{(Eq. 3.5f)}$$

$$c=2.50R+14.0\frac{\sigma_{ck}}{\sigma_y}-2.50 \quad \text{(Eq. 3.5g)}$$

where m_y is the yield moment, ϕ_y is the curvature, corresponding to the yield moment. and m_{pc} is the flow moment.

The width-thickness ratio parameter, R, is defined in (Eq. 3.6). Since the buckling coefficient k cannot be calculated directly, the parameter was decided using the buckling stress σ_{cr} obtained by the elastic buckling analysis.

$$R=\sqrt{\frac{\sigma_y}{\sigma_{cr}}}=\frac{b}{t}\sqrt{\frac{12(1-\nu^2)}{\pi^2 k}}\sqrt{\frac{\sigma_y}{E}} \quad \text{(Eq. 3.6)}$$

The results obtained by Usami's moment-curvature relation are shown and compared with Uy's test results in Fig. 8. The results are in solid agreement with the elastic range of the material but there is a significant discrepancy in the plastic range. This disagreement seems to stem from the different material properties that exist between the cold-formed and hot-rolled steel sections. (Eq. 3.5d-f) are modified and simplified as (Eq. 3.7a-c) to revise the differences taking into consideration that the axial force equals zero.

$$m_y=0.128R+0.281 \quad \text{(Eq. 3.7a)}$$

$$\phi_y=0.56 \quad \text{(Eq. 3.7b)}$$

$$m_{pc}=1.0 \quad \text{(Eq. 3.7c)}$$

The results obtained by the modified version of Usami's formula are given in Fig. 8 for comparison with other test results and Usami's results. As shown in Fig. 8, the predicted results by the modified formulas are in agreement with the test results. The local buckling of the steel plate, above top rip, cannot be included in this formula. However, the occurrence of local buckling of the steel plate does not affect the ultimate flexural rigidity of the composite sections, since the post-buckling stage is stable in the local buckling mode.

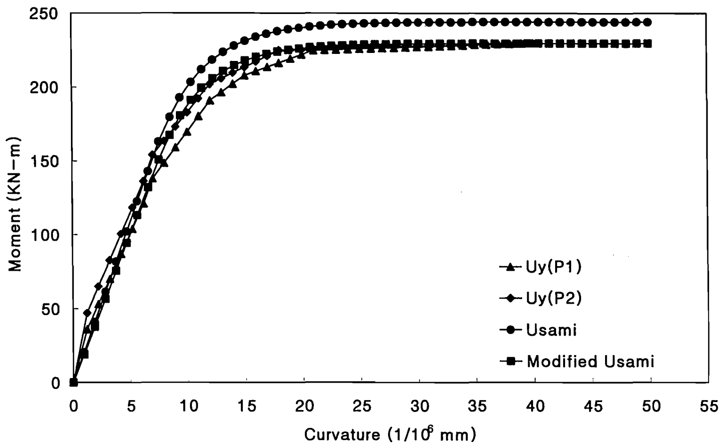


Fig. 8. Moment-Curvature Relation by the Modified Usami Model

Power Model

Several power models have been applied to predict the moment-rotational behavior of the beam to column connections. In defining the moment-curvature relation of the composite beam from the test results, both the Cubic B-Spline Model and the Power Model are useful. Though the Cubic B-Spline Model can give accurate curves, its' application to the moment-curvature curve is difficult because of the large volume of data that is needed. On the other hand, the Power Model is easily applicable, in order to obtain a comparatively, accurate curve fit with less data. Therefore, the Power Model has been adopted in this paper to explain the moment-curvature relation of the profiled composite beam.

The Power Model has been used to derive the moment-rotational relation of beam to column connections from the test results. The Power Model has been utilized by Colson and Louveau(1983), Kishi and Chen(1990), and Ang and Morris(1984). In this paper, Kishi and Chen's model was selected. The model proposed by Kishi and Chen needed three parameters, i.e. initial connection stiffness, ultimate moment capacity and shape parameters.

The Power Model is expressed in Eq. 3.8a, and b where the initial connection stiffness is substituted by the slope of the moment-curvature curve and the ultimate moment capacity is replaced by the full plastic moment.

$$R_{ki} = \frac{M_{pc,0} \cdot a_{00}}{\Phi_y} \quad \text{(Eq. 3.8a)}$$

$$M_{pc,0} = \frac{1}{2}(b-t)\,\eta_0^2\,\sigma_{ck} + [\,(b+t)\,d + \eta_0^2 + (d-t-\eta_0)^2]\,t\,\sigma_y \quad \text{(Eq. 3.8b)}$$

$$\text{where } a_{00} = \frac{(1.0347R+0.2905)p+(0.13339R+0.28615)}{0.7070p+0.5329} \quad \text{(Eq. 3.8c)}$$

$$\eta_0 = \frac{2(d-t)t\sigma_y}{(b-t)f_{ck}+4t\sigma_y} \quad \text{(Eq. 3.8d)}$$

The symbols Φ_y, $M_{pc,0}$ and a_{00} are the variables which were derived to trace the moment-curvature relations by Ge and Usami. However, in considering that the axial force equals zero and the material properties of the steel used were different, the parameter a_{00} was modified as:

$$a_{00} = \frac{0.128R+0.281}{0.35} \quad \text{(Eq. 3.9)}$$

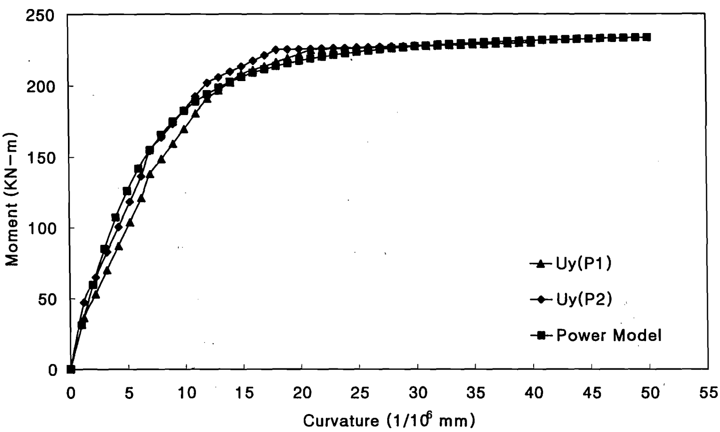


Fig. 9. Moment-Curvature Relation by the Power Model

The moment-curvature of the composite beam can be expressed as Eq. 3.10 using R_{ki}, $M_{pc,0}$ and shape parameter n.

$$M = \frac{R_{ki} \cdot \phi}{\left\{1+\left(\frac{\phi}{\phi_0}\right)^n\right\}^{\frac{1}{n}}} \quad \text{(Eq. 3.10)}$$

The results obtained by the proposed Power Model are shown in Fig. 9, where the shape parameter n was taken as 1.7, and the results were compared with Uy's test results. As shown in Fig. 9, the model can produce an accurate moment-curvature relation of the composite beam which is composed of cold-formed sheeting and concrete. The model gave a slightly stiffer slope than the test results after local buckling occurred as in the modified Usami Model.

PARAMETRIC STUDY

A parametric study has been undertaken using the Power Model to discover the effects of thickness and strength of the cold-formed steel section and the concrete cube strength.

Effects of Thickness of Cold-Formed Steel Section

The thickness of the cold-formed steel section was varied. The adopted gauge thicknesses were 0.8, 1.0, 1.5mm. The results of the variation of thickness are shown in Fig. 10a. The stiffness and strength of the composite section were significantly increased by increasing the thickness, as shown.

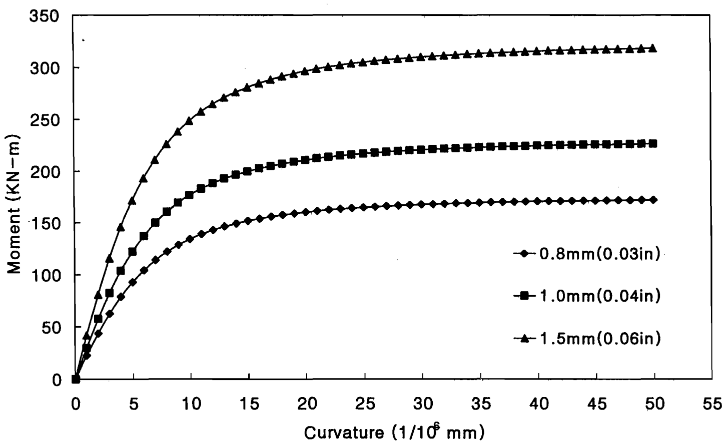


Fig. 10(a). Effects of Cold-Formed Steel Thickness

Effects of Cold-Formed Steel Strength

The effect of the yield stress of cold-formed steel sections was studied for the chosen values of σ_p= 320, 440, 552 MPa. Fig. 10b shows the moment-curvature relations of the composite beam with various yield stresses. The change of the initial stiffness of the composite beam was not significant but the variation became more produced when the moment is increased. The ultimate strength was increased substantially with the increase of the yield stress of the steel sheeting.

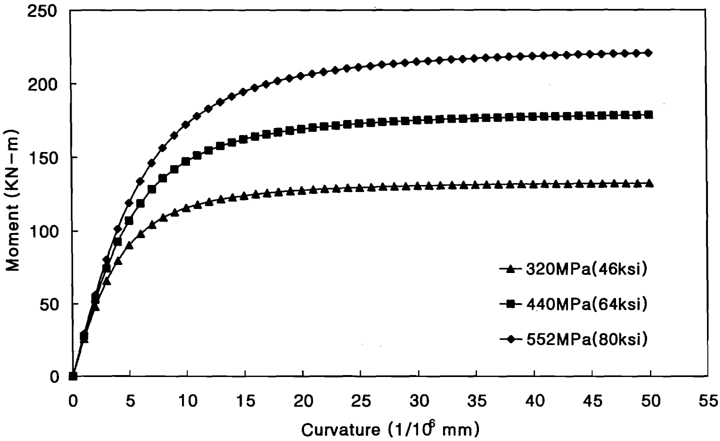


Fig. 10(b). Effects of Cold-Formed Steel Strength

Effects of Concrete Cube Strength

The concrete cube strength σ_{ck} was varied for the values of 27.7, 44.2 and 68.3 MPa. The effect on the strength of the composite beam has been investigated. Both stiffness and strength of the composite beam were slightly increased with the increase of the concrete cube strength, as shown in Fig. 10c. The consequence can be explained by the fact that the strength change of the concrete affects the compressive zone while the change of the steel sheeting affects both tensile and compressive areas. Consequently, the use of high strength concrete has little merit and is not recommended.

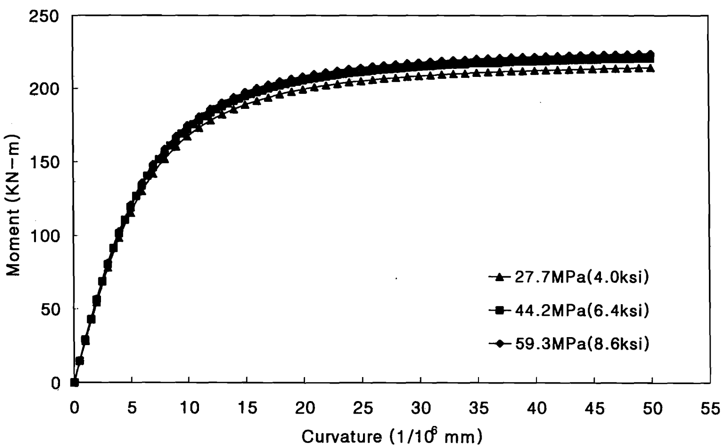


Fig. 10(c) Effects of Concrete Cube Strength

LOAD-DEFLECTION BEHAVIOR OF COMPOSITE BEAMS

The nonlinear load-deflection relation can be obtained simply by the numerical method, using the known moment-curvature relations. The step-by-step numerical integration method, with the previous moment-curvature relation model, is developed to trace the load-deflection relation of the profiled composite beam. In the step-by-step numerical integration procedure, the member is divided into segments in a longitudinal direction. Each of the division points is referred to as a "station". The deflection of the first station is specified. The deflections at subsequent stations are calculated station by station in a systematic routine procedure. The assumed deflection of the first station is kept constant during the iteration and the forces that correspond to the deflection are calculated. The solution procedure for a simply supported beam is summarized as follows:

① Specify a deflection at station 1 and assume a value for M_1.

② Calculate the secondary moment and then total moment at station 1

③ Obtain the curvature at station 1 from the moment-curvature relation.

④ Calculate the deflection at each station beginning of the second station using

$$y_s = -\phi_{s-1}(\Delta x)^2 + 2y_{s-1} - y_{s-2}$$

where y_s : deflection at station s

ϕ_{s-1} : curvature at station s-1

Δx : distance between stations

⑤ Calculate the moment and curvature at stations up to the last station of the beam.

⑥ If the deflection at the last station of the member differs from zero, a correction is made to M_1 by

$$M_2 = (1 + \frac{1}{n}\frac{y_n}{y_1})M_1$$

where n represents the number of segments.

The process ②-⑥ is repeated with M_2 until the deflection at the last station becomes zero.

⑦ The process ①-⑥ is repeated until the required moment-deflection curve is obtained.

The results obtained by above mentioned procedures are shown and compared with Uy's test results in Fig. 11. The comparison shows that the Power Model produces slightly better results than the modified Usami Model. However, appropriately, both models for the moment-curvature relation can be used to predict the nonlinear load-deflection behavior of profiled composite beams.

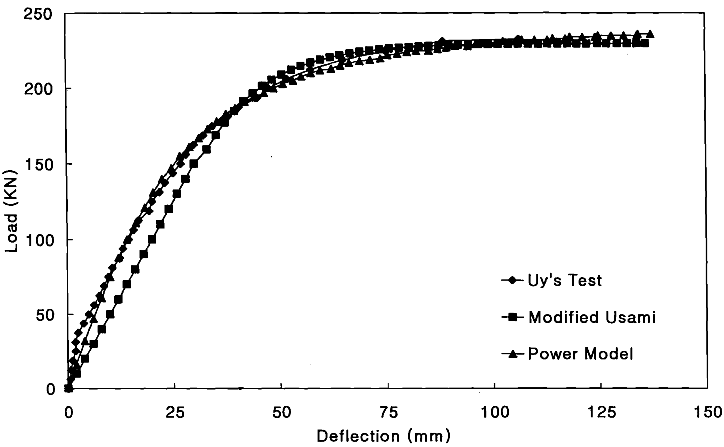


Fig. 11. Load-Deflection of a Composite Beam

CONCLUSION

An analytical method to trace the nonlinear moment-curvature relation of the composite beam has been developed to include the nonlinear material properties of steel sheeting, reinforcing steel bar and concrete. The method can produce quite reasonable numerical results in comparison with other test results.

Simple formulas, which can be used to predict the moment-curvature relation of the profiled composite beams, have also been proposed. Though both the Power Model and the Modified Usami Model produce quite a good moment-curvature relation, in comparison with the test results, the Power Model is slightly better for simulation of the nonlinear load-deflection behavior of the composite beam. With simple modification of the formulas proposed, the method can be easily applied to investigate the structural behavior of composite beam-columns. The step-by-step numerical integration method with the proposed moment-curvature models has been proposed to simulate the load-deflection relation of the profiled composite beam and has been proven to be accurate and effective. However, the methods proposed, need to be verified further, by comparison with various test results.

The change of the properties of the steel sheeting has a significant effect on the stiffness and strength of the composite beam. It was found that changes of concrete strength have little effect on the structural behavior of composite beams.

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